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Combined Treatment of Domestic and Industrial Wastes by Activated Sludge

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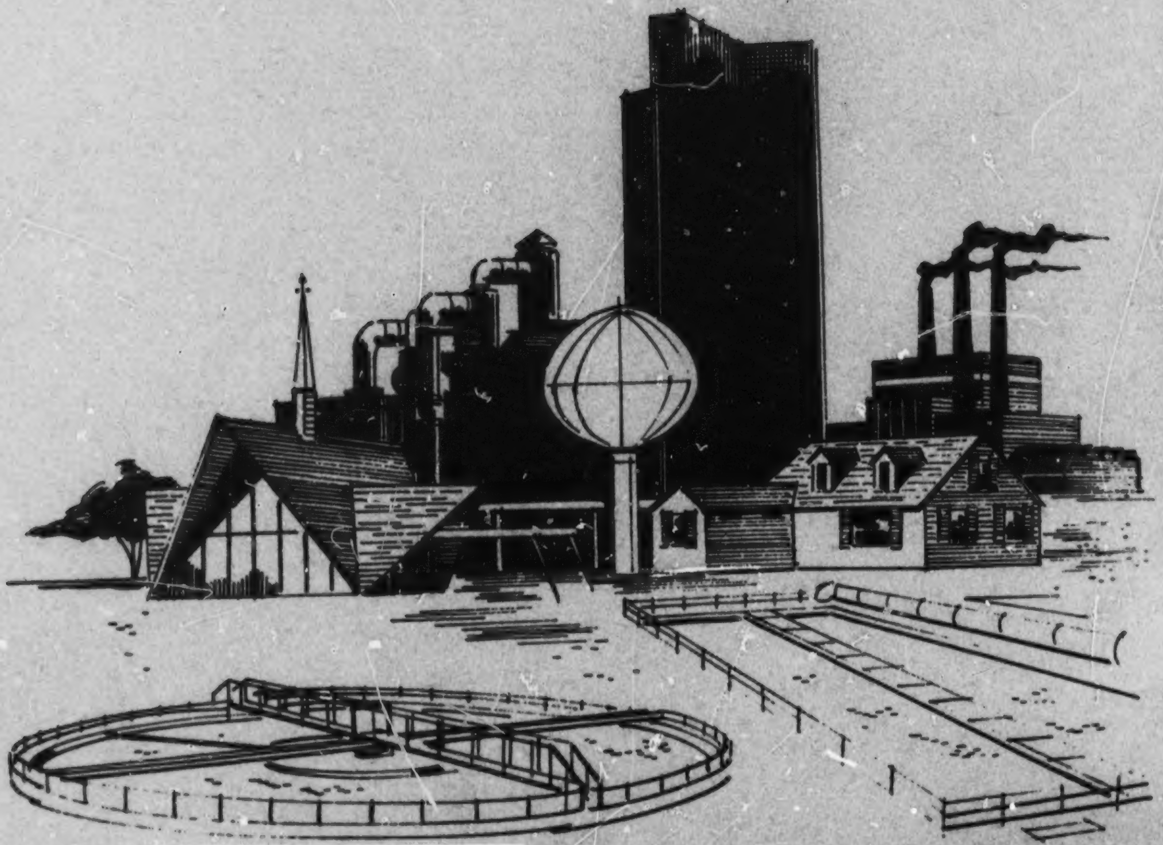

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16. Abstract A completely aerobic secondary treatment facility was constructed to treat both domestic wastes from the City of Dallas, Oregon, and cannery wastes in season as well. The system was designed for 2.0 mgd average daily flow and a BOD loading of 7000 lbs/day, and its operation was monitored from August 1969 to November 1970. Results indicated that: (1) start-up, operation, and shut-down of the cannery caused no operational problems; (2) the treatment system was sufficiently flexible and stable to withstand shock organic and hydraulic loadings; and (3) a good quality effluent was produced with aeration times varying from 4 to 60 hours. Average effluent BOD and suspended solids concentrations were 8 and 13 mg/l respectively for the test period. Biological solids yield averaged 0.7 lbs solids/lb BOD removed, with a net accumulation of .42 lbs solids/lb BOD removed. MLSS concentration ranged from 700 to 3000 mg/l, average sludge age was 19 days, and the organic loading range was from .05 to .40 lbs BOD/lb MLSS/day. Total capital cost was 66% of that for a conventional activated sludge system, and operation and maintenance costs were only 33% of those for a conventional system. (Lowry-Texas)

17a. Descriptors *Waste water treatment, *Municipal wastes, *Industrial wastes, Canneries, Activated sludge, Operation and maintenance, Organic loading, Aeration, Cost analysis, Oregon

17b. Identifiers *Aerobic digestion, *Process stability, *Process flexibility, Shock loads, *Dallas (Oregon)

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Combined Treatment of Domestic and Industrial Wastes by Activated Sludge

by

The City of Dallas, Oregon

for the

WATER QUALITY OFFICE
ENVIRONMENTAL PROTECTION AGENCY

Grant No. 11060 EZR

Project No. 12130 EZR

MAY 1971

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This report has been reviewed by the Environmental Protection Agency and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

ABSTRACT

The operation of a completely aerobic secondary treatment facility for treatment of combined domestic and industrial wastewater from the City of Dallas, Oregon, was studied for a period of 15 months. The system was designed for an average daily flow of 2.0 mgd and a BOD load of 7000 pounds per day. The results of this study indicate the flexibility and economy of the completely aerobic system, consisting of activated sludge with aerobic digestion, for a small community with proportionately high industrial wastewater loads. The effluent BOD concentration averaged 8 mg/l and the effluent total suspended solids concentration averaged 13 mg/l for the 15-month study period. The biological solids yield averaged about 0.7 pounds of solids per pound of BOD removed and the net accumulation of biological volatile solids was about 0.42 pounds of volatile solids per pound of BOD removed. These values were obtained with a MLSS concentration range of 700 to 3000 mg/l, an average sludge age of 19 days and an organic loading range of 0.05 to 0.40 pounds of BOD per pound of MLSS per day. Total capital cost of the system was about 66 percent of that for a conventional activated sludge plant and operation and maintenance costs were only about 33 percent of those for a conventional system.

This report was submitted in fulfillment of Grant No. 11060 EZR under the partial sponsorship of the Water Quality Office, Environmental Protection Agency.

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CONTENTS

SECTION		PAGE
I	CONCLUSIONS	1
II	RECOMMENDATIONS	3
III	INTRODUCTION	5
	Scope	5
	Background	6
	Theoretical Considerations	7
IV	TREATMENT FACILITIES	11
	General Description	11
	Design Concept	14
	Design Criteria	14
	Design Factors	15
V	DEMONSTRATION PROCEDURES	23
	Plant Startup	23
	Operation	23
	Sampling Schedule and Procedures	23
	Analytical Methods	26
VI	WASTEWATER CHARACTERISTICS	29
	General	29
	Canning Season	29
	Dry Weather Non-Canning Season	34
	Wet Weather Non-Canning Season	34
	Infiltration	35
VII	TREATMENT PLANT PERFORMANCE	37
	General	37
	Effluent Quality and System Stability	37
	Aerobic Digestion	47
	Solids Deposits	51
	Velocity Profiles	51
	Dissolved Oxygen Profiles	57
	Oxygen Uptake Rates	57
	PVC Liner	57
	Operational Considerations	62

CONTENTS - CONTINUED

SECTION		PAGE
VIII	FINANCIAL CONSIDERATIONS	63
	Construction Costs	63
	Operation and Maintenance Costs	63
	Total Annual Costs	63
IX	DISCUSSION	65
	Activated Sludge System	65
	Aerobic Digestion	68
	Chlorine Contact	75
X	ACKNOWLEDGMENTS	77
XI	REFERENCES	79
XII	PUBLICATIONS	81
XIII	ABBREVIATIONS	83
XIV	APPENDIXES	85

FIGURES

NO.		PAGE
1	Treatment Facilities	12
2	Schematic Plan	13
3	Aeration Basin, Typical Section	18
4	Aerobic Digester, Typical Section	20
5	Chlorine Contact Channel, Typical Section	21
6	Flow and Rainfall vs. Time	31
7	Influent BOD vs. Time, Weekly Average Data	32
8	BOD Loading (lb/day) vs. Time, Weekly Average Data	33
9	Influent and Effluent BOD vs. Time, Weekly Average Data	38
10	Influent and Effluent TSS vs. Time, Weekly Average Data	39
11	Influent BOD and Effluent Total and Soluble BOD vs. Time, Canning Season	43
12	BOD Removal vs. Time, Canning Season	44
13	Influent BOD and Effluent Total and Soluble BOD vs. Time, Dry Weather Non-Canning Season	45
14	Influent BOD and Effluent Total and Soluble BOD vs. Time, Wet Weather Non-Canning Season	46
15	Effluent Soluble BOD vs. Aeration Time	48
16	Organic Loading vs. Time, All Data	49
17	Solids Accumulation in Aeration Basin No. 1, October 1970	52
18	Aeration Basin Velocity Profile, Two Aerators, Low Speed (one foot)	53

FIGURES – CONTINUED

NO.		PAGE
19	Aeration Basin Velocity Profile, Two Aerators, Low Speed (6 feet)	54
20	Aeration Basin Velocity Profile, Two Aerators, Low Speed (9 feet)	55
21	Aeration Basin, Velocity Profiles	56
22	Aeration Basin D. O. Profile, Two Aerators, Low Speed (one foot)	58
23	Aeration Basin D. O. Profile, Two Aerators, Low Speed (6 feet)	59
24	Aeration Basin D. O. Profile, Two Aerators, Low Speed (9 feet)	60
25	Aeration Basin D. O. Profiles	61
26	Substrate Removal vs. Effluent Soluble BOD	66
27	Clarifier Hydraulic Loading vs. Effluent Suspended Solids	69
28	Clarifier Solids Loading vs. Effluent Suspended Solids	70
29	Horsepower – Oxygen Transfer Relationships	71
30	Composition of Waste Sludge Solids	73

TABLES

NO.		PAGE
1	Treatment Plant Design Criteria	14
2	Design Factors	16
3	Detailed Operation Schedule	24
4	Sampling and Testing Schedules	25
5	Influent Characteristics	30
6	Effluent Characteristics	40
7	Influent and Effluent Characteristics, All Data	41
8	Total Annual Costs	64

SECTION I
CONCLUSIONS

Operation of a completely aerobic wastewater treatment plant, designed to provide secondary treatment of domestic and industrial wastes from the City of Dallas, Oregon, has been studied during the period from August, 1969 through November 1970.

The following conclusions have been reached, based on the results of the study presented in this report:

1. Effluent quality was good throughout the study period, even during such extreme conditions as periods of flow at maximum hydraulic capacity during the non-canning season.
2. Cannery startup, operation and shutdown have had no significant effect on the operation of the treatment system.
3. The system is sufficiently stable to produce a good quality effluent with aeration times varying from 4 hours to 60 hours.
4. The system has adequate flexibility and stability to withstand shock organic and hydraulic loads.
5. The wastewater characteristics during the first year of operation more closely resembled domestic wastewater as opposed to industrial wastewater characteristics.
6. Effluent BOD averaged 8 mg/l and effluent suspended solids averaged 13 mg/l, for the entire study period.
7. The substrate removal coefficient, k, averaged about 0.041 and was not significantly affected by changes in the temperature range of data encountered.
8. The biological sludge yield averaged about 0.7 pounds of solids per pound of BOD removed.
9. Net biological volatile solids accumulation was about 0.42 pounds of volatile solids per pound of BOD removed.

10. The biodegradable portion of the solids wasted to the digester averaged about 19 percent of the total wasted solids.

11. Aerobic digestion is an effective and economical means of waste activated sludge stabilization.

12. The average reduction in volatile solids content through the digester was 5 percent.

13. Digested sludge has been stabilized to the extent that no objectionable odors have evolved from the humus storage ponds.

14. A treatment system of this type can be constructed and operated at substantial savings in cost compared to a conventional activated sludge system.

15. Total capital cost was about 66 percent of the cost for a conventional 2 mgd activated sludge plant.

16. Operation and maintenance costs for the first year averaged $33 per million gallons treated, or about 33 percent of those for a conventional plant treating the same average flow.

17. Total annual treatment cost averaged $0.097 per pound of BOD removed (amortized capital cost portion of annual cost based on BOD removal capacity rather than actual pounds of BOD removed).

SECTION II
RECOMMENDATIONS

Analysis of the data obtained during the demonstration period indicates several areas where design modifications or further operational studies would be desirable.

DESIGN MODIFICATIONS–Only minor design modifications are recommended and are as follows:

The concrete protective pads beneath the aerators should be larger to prevent bottom scour.

Measurement of recycled sludge flow with a propeller type meter is not satisfactory because of frequent fouling of the propeller. A magnetic flowmeter would be preferred.

FUTURE STUDIES–Organic loadings to the plant during this study were low, relative to design capacity. Analysis of some aspects of the treatment unit performances was very difficult to achieve because of this. Therefore, it would be desirable to reevaluate some of these analyses at loads closer to the design capacity. The extremely low solids loading rate and the low degradability of the solids wasted to the digester during this study precluded an accurate analysis of digester performance under a full range of operating conditions. Future studies of the performance of the aerobic digester would be particularly useful.

SECTION III
INTRODUCTION

SCOPE

A completely aerobic system designed to provide secondary treatment of combined domestic and industrial wastes was constructed at the City of Dallas, Oregon, and was studied during the period from September 1969 through November 1970. This project was financed with the aid of a research, development and demonstration grant provided by the Environmental Protection Agency (EPA), under grant number 11060 EZR.

The basic purpose of this program was to demonstrate the economics and efficiency of the completely aerobic treatment method when applied to the treatment of combined domestic and industrial wastes from a small municipality.

The specific objectives were to:

1. Demonstrate, in full-scale plant operation, the BOD and suspended solids removal capabilities and stability of the completely mixed, completely aerobic system treating combined domestic and industrial waste flows from a small municipality.

2. Determine the influence of rapidly changing industrial waste loadings resulting from startup and shutdown of canning operations, changes in canning product, and brief periods of cannery inactivity during the canning season.

3. Determine the efficiency and operating characteristics of the combined treatment system during the off-canning season.

4. Determine the effect of varying the detention time in the aeration basins, between 12 and 48 hours, on BOD and suspended solids removal and system stability.

5. Demonstrate the applicability of aerobic digestion to biological treatment systems serving small municipalities.

6. Study the quantity and character of aerobically digested sludge when varying the liquid and solids detention time in the aerobic digester between 5 and 20 days.

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7. Demonstrate the savings in both construction and operating costs of the completely aerobic treatment method when utilizing earthen basin construction and floating mechanical surface aeration units for mixing and supplying oxygen to the aeration basins and aerobic digester.

8. Demonstrate the use of polyvinyl chloride (PVC) sheet lining in waste treatment plant construction.

BACKGROUND

Many small cities are faced with the problem of providing wastewater treatment services to local industry. Waste loads from such industries as food processors are seasonal in nature and may constitute the major organic load on the wastewater treatment plant. These small cities therefore require a treatment system which will withstand these shock loads, yet is economical enough to minimize the financial burden on the community. The completely aerobic system was evaluated on this basis.

The City of Dallas, Oregon, retained the engineering firm of Cornell, Howland, Hayes & Merryfield to design and supervise the construction of the treatment facility and to exercise technical supervision over the research and development program.

An EPA research and development grant (No. 11060 EZR) was accepted by the City on 21 February 1968.

The treatment system receives a combination of domestic and commercial wastes, cannery wastes, a limited quantity of slaughterhouse wastes and an occasional flow of phenolic glue waste from a plywood mill.

The plant was designed to meet water quality requirements established by the Oregon State Department of Environmental Quality, which state that effluent to the receiving stream may contain not more than 20 milligrams per liter (mg/l) each of BOD and suspended solids. The process employed was a completely mixed activated sludge system with aerobic digestion of the waste solids.

A complete list of definitions of the technical terms used in this report may be found in the WPCF Glossary [1], and a list of abbreviations and symbols used is contained in Section XIII.

THEORETICAL CONSIDERATIONS

MICROBIOLOGY—The living organisms found in activated sludge are classified as either plants or animals. The plants consist of bacteria and fungi and the animals are primarily protozoa, rotifers and nematodes.

Hawkes [2] stated that bacteria are normally dominant as primary feeders on organic wastes, with different holozoic protozoa being secondary feeders, and rotifers and nematodes are found at the higher levels in the food chain. Fungi cannot normally compete with bacteria, but they may predominate as primary feeders if certain conditions exist, such as: low pH, nitrogen deficiency, or low dissolved oxygen [3]. High-carbohydrate wastes are also reported to stimulate fungi growth.

The composition of the organic waste determines which bacterial genera will predominate [2,3]. Protein wastes favor *Alcaligenes, Flavobacterium, and Bacillus,* while carbohydrate waste favors *Pseudomonas.* A high population of free swimming bacteria will sustain free swimming ciliata as the predominate protozoa; however, if the food level is lowered by a reduction in the free swimming bacterial population, the free swimming ciliates will yield to stalked ciliates which require less energy.

Rotifers thrive in very stable systems and are better indicators of stable conditions than are the nematode worms.

METABOLISM—The metabolic reactions which occur within activated sludge can be divided into three phases: (1) oxidation, (2) synthesis, and (3) endogenous respiration. These three phase reactions can be illustrated with the following general equations, which have been formulated by Weston and Eckenfelder [4]:

(1) Organic Matter Oxidation

$$C_xH_yO_z + aO_2 \rightarrow xCO_2 + bH_2O + \text{Energy}$$

$$\text{where } a = x + \frac{y}{4} - \frac{z}{2}$$

$$b = \frac{y}{2}$$

(2) Cell Material Synthesis

$$C_xH_yO_z + NH_3 + dO_2 + \text{Energy} \rightarrow C_5H_7NO_2 + eCO_2 + fH_2O$$

where $d = \frac{x + y}{4} - \frac{z}{2} - 5$

$e = x - 5$

$f = \frac{y}{2} - 2$

(3) Cell Material Oxidation

$$C_5H_7NO_2 + 5O_2 \rightarrow 5CO_2 + 2H_2O + NH_3 + \text{Energy}$$

In the presence of enzymes, produced by living microorganisms, about one-third of the organic matter removed is oxidized to carbon dioxide and water, to provide energy for synthesis to cell material of the other two-thirds of the organic matter removed [5]. The cell material is also oxidized to carbon dioxide, water, etc., by endogenous respiration (auto-oxidation).

KINETICS—Several authors [6,7,8] have formulated mathematical equations for design and operation of complete-mix activated sludge plants. Some of these formulations are more easily used for evaluation of full-sized plant operation than others. Eckenfelder's basic equations [7,9] are of this nature and are presented below.

SUBSTRATE UTILIZATION—The Michaelis-Menton relationship was used to define the microbial growth rate and steady state substrate removal in a completely mixed system, and a simplified equation for substrate removal was developed:

$$kS_e = \frac{S_a - S_f}{X_a t} = \frac{S_r}{X_a t}$$

where S_r = BOD removed, lbs per day
S_a = influent BOD, lbs per day
S_f = soluble effluent BOD, lbs per day
S_e = soluble effluent BOD, mg/l
X_a = average mixed liquor volatile suspended solids, lb
t = aeration time, days
k = removal rate coefficient (lb BOD/day/lb MLVSS) per mg/l BOD

The equation shows that the substrate removal is proportional to the product of the MLVSS and the aeration time. This equation is valid only for conditions in which the actual substrate concentration is much less than the substrate concentration at one-half the maximum reaction rate (Michaelis Constant). A low effluent soluble BOD from a completely mixed system indicates a low substrate concentration since the effluent soluble BOD is the same as the soluble BOD in the aeration basin. Therefore, the above condition is met if the effluent soluble BOD is consistently low.

SLUDGE YIELD–Excess solids in the activated sludge system will result from the non-biodegradable suspended solids in the influent and the biological cells synthesized in the system during BOD removal, less the quantity of cell mass synthesized which is broken down by endogenous respiration.

McCarty and Brodersen [10] have presented an equation which gives the net sludge accumulation in terms of pounds of volatile biological solids produced per pound of BOD removed, as follows:

$$\frac{A}{F} = 0.65 - \frac{0.53}{1 + \frac{E}{bM}}$$

where A = net accumulation of volatile biological solids
F = BOD removed
E = suspended solids lost from system per day
M = total suspended solids in system
b = endogenous respiration constant

This equation shows that the net accumulation of solids is dependent on the rate of synthesis of biological solids, the rate of solids degradation by endogenous respiration and the sludge age of the system.

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SECTION IV
TREATMENT FACILITIES

GENERAL DESCRIPTION

Figure 1 is an aerial photograph of the treatment system. The photograph was taken just prior to completion of construction.

Waste flow from the City's collection system enters the plant through a 36-inch diameter influent sewer at the pump station (1). The raw waste is pumped through an 18-inch pipeline to the headworks (2). The waste is shredded by the comminutor (3) and then flows to the aeration basin splitter box (4) where it is discharged to one or both aeration basins (5). The effluent from the aeration basin flows by gravity to the clarifier (6) where the activated sludge is settled.

The activated sludge to be returned to the aeration basin is removed from the clarifier by means of six suction pipes mounted on the revolving clarifier mechanism and flows by gravity to the return sludge pump station located nearby (7). Two sludge pumps, sized to return 25 to 100 percent of the plant dry weather design flow, lift the activated sludge back up to the splitter box (4).

Excess, or waste, sludge is collected on the bottom of the clarifier by a revolving scraper arm and pumped directly to the aerobic digester (8) through sludge pumps located in the control building (9).

The effluent from the clarifier flows by gravity through the flow measurement box (10) to the chlorine contact channel (11) and is finally discharged through the outfall (12) into LaCreole Creek.

A sludge pump located in the control building transfers the digested sludge to one, or both, humus storage ponds (13) where it is further stabilized and dried. The supernatant from the humus ponds flows by gravity to the plant influent line (1).

Figure 2 is a schematic diagram of the treatment system, showing flow pattern and locations of sampling points, flow metering, control valves and gates, and pumps.

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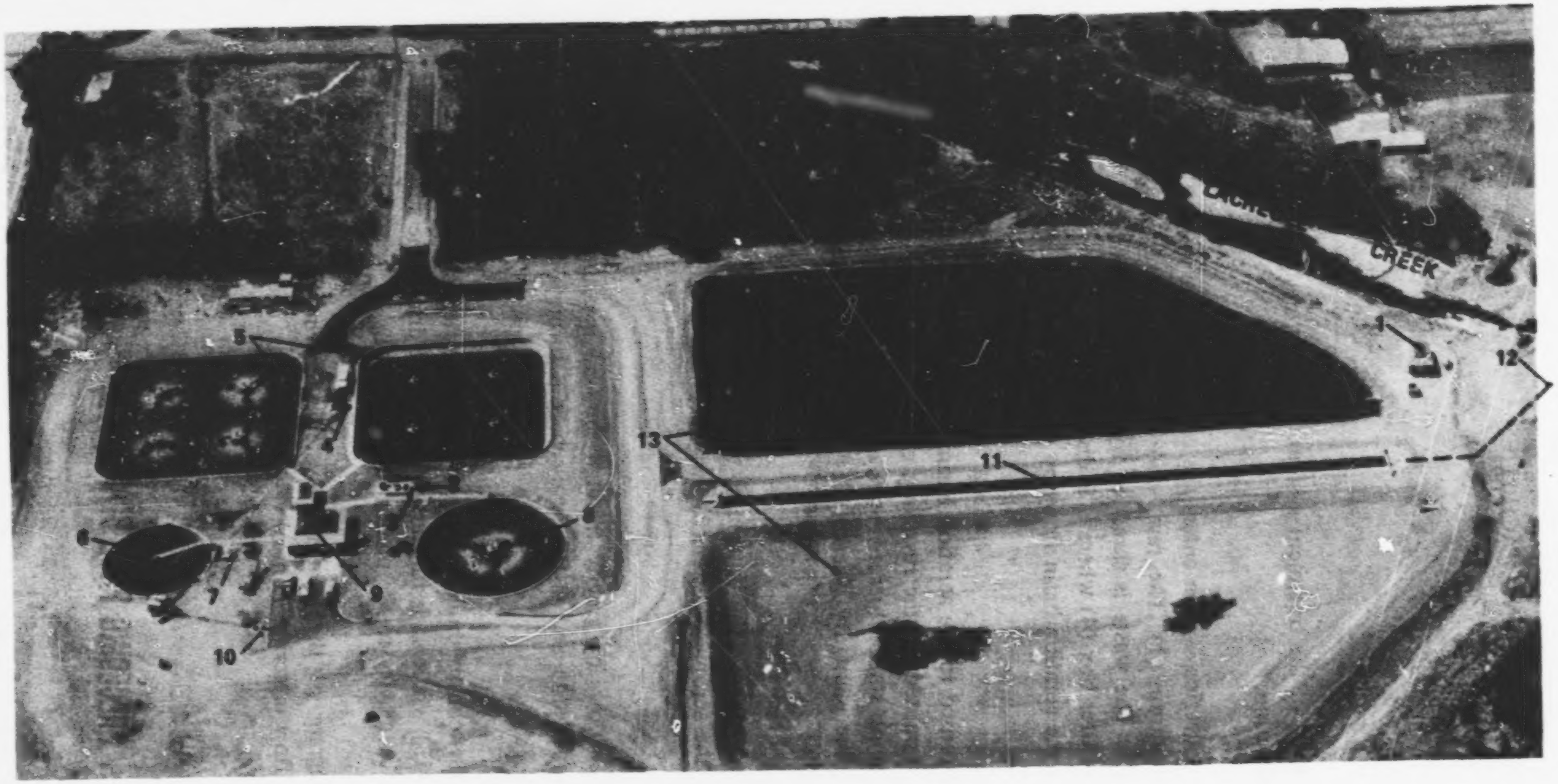


1. RAW SEWAGE PUMP STATION
2. HEADWORKS
3. COMMINUTOR
4. AERATION BASIN SPLITTER BOX
5. AERATION BASINS (2)
6. FINAL CLARIFIER
7. RETURN SLUDGE PUMP STATION
8. AEROBIC DIGESTER
9. CONTROL BUILDING
10. FLOW MEASUREMENT BOX
11. CHLORINE CONTACT CHANNEL
12. OUTFALL SEWER
13. SLUDGE LAGOONS (2)

NOT REPRODUCIBLE

FIGURE 1

TREATMENT FACILITIES

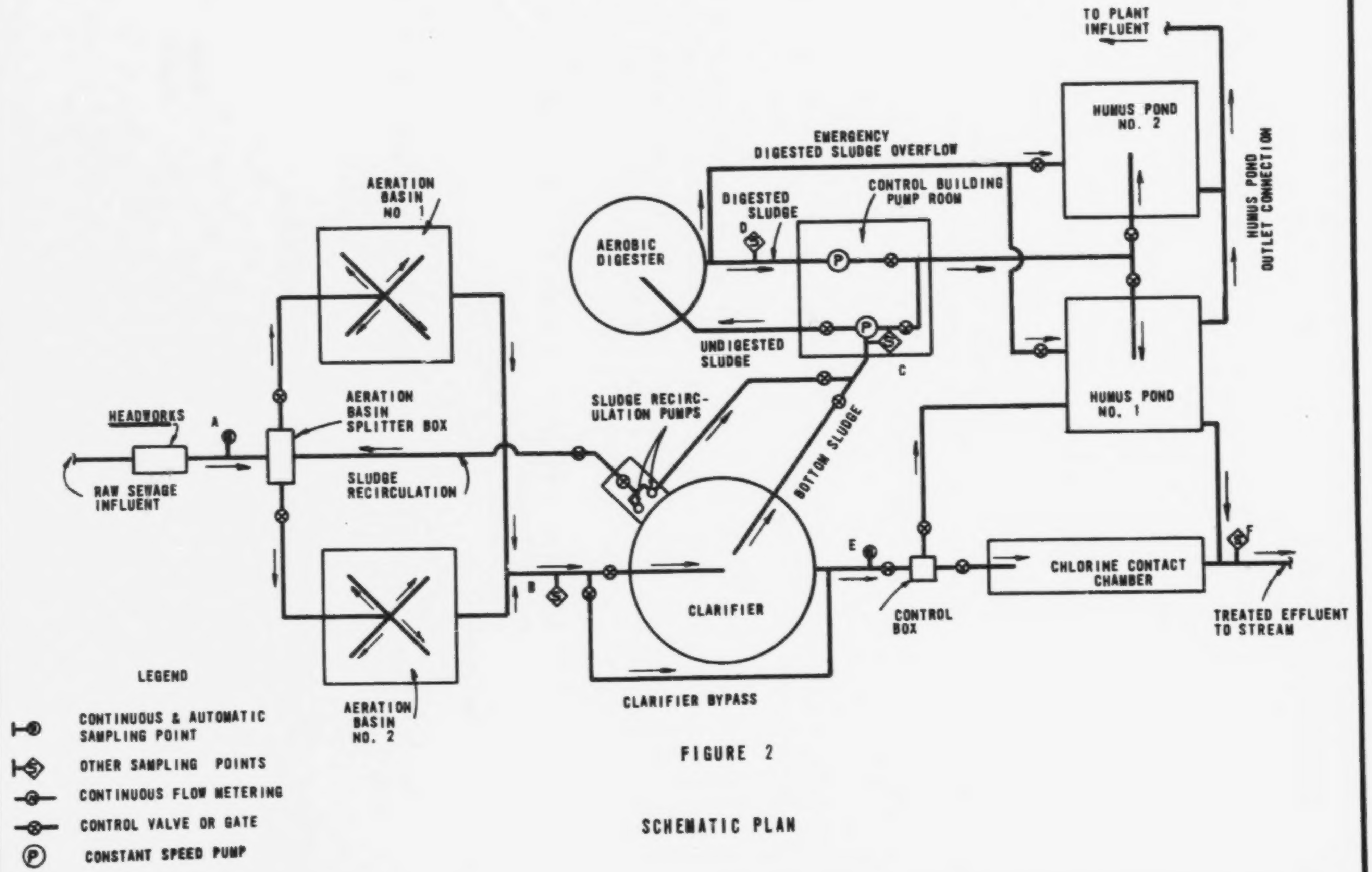

TO PLANT INFLUENT
HUMUS POND NO. 2
HUMUS POND OUTLET CONNECTION
EMERGENCY DIGESTED SLUDGE OVERFLOW
AERATION BASIN NO 1
DIGESTED SLUDGE
CONTROL BUILDING PUMP ROOM
D
AEROBIC DIGESTER
P
UNDIGESTED SLUDGE
C
HUMUS POND NO. 1
HEADWORKS
A
AERATION BASIN SPLITTER BOX
SLUDGE RECIRCULATION
SLUDGE RECIRC-ULATION PUMPS
BOTTOM SLUDGE
RAW SEWAGE INFLUENT
B
E
F
CLARIFIER
CHLORINE CONTACT CHAMBER
CONTROL BOX
TREATED EFFLUENT TO STREAM
AERATION BASIN NO. 2
CLARIFIER BYPASS
LEGEND
CONTINUOUS & AUTOMATIC SAMPLING POINT
OTHER SAMPLING POINTS
CONTINUOUS FLOW METERING
CONTROL VALVE OR GATE
CONSTANT SPEED PUMP

FIGURE 2

SCHEMATIC PLAN

DESIGN CONCEPT

The design was based on the need to minimize the following shortcomings often associated with conventional activated sludge designs: high construction costs; high operating labor costs; need for highly trained operators; susceptibility to upset by shock organic, hydraulic and toxic loads; and waste solids handling, stabilization and disposal problems.

These shortcomings were minimized by use of the following design concepts:

Primary clarification and grit removal facilities were eliminated. Grit will settle in the aeration basins and will be removed periodically by heavy equipment. The aeration capacity was increased to compensate for the greater organic load to the aeration basins.

Aerobic digestion was employed in place of the higher cost conventional anaerobic digestion.

Earthen basins, with shotcrete sideslope linings, were used in place of higher cost reinforced concrete construction.

The complete-mix activated sludge principle, designed for a conservative range of organic loadings and adequate hydraulic surge capacity, was employed to reduce susceptibility to process upset. Hydraulic surge capacity was provided by the use of narrow aeration basin outlets which provide a more uniform basin effluent flow, and allowance for a fluctuation of about one foot in the basin water level.

The use of aerobic digestion and automatically controlled sludge pumping simplifies waste solids handling and disposal problems.

DESIGN CRITERIA

The criteria used for final design of the City of Dallas wastewater treatment plant are listed in Table 1.

TABLE 1
TREATMENT PLANT DESIGN CRITERIA

POPULATION	
Present	5,900
Design	10,400

FLOW (MGD)	
Present Average	0.94
Design Average	2.00
Design Maximum, day	6.00
Peak Instantaneous	7.00
BOD LOAD (LB/DAY)	
Canning Season	7,080[1]
Non-Canning Season	2,080
Population Equivalent[2]	
Canning Season	41,600
Non-Canning Season	12,200
EFFLUENT REQUIREMENTS	
Maximum BOD (mg/l)	20
Maximum S.S. (mg/l)	20

[1] Includes 5,000 lb/day industrial BOD

[2] Based on 0.17 lb BOD/capita/day

DESIGN FACTORS

The design factors used for the major unit and equipment selection are listed in Table 2. Photographs of several of the major units are included in Appendix A. Appendix B lists the manufacturers of the major equipment items. The major treatment units are described below.

Each aeration basin provides a volume of 1.0 million gallons. The basins were constructed of earth with a reinforced concrete ring wall and shotcrete lined sideslopes. A typical section through the aeration basin is shown on Figure 3. Oxygen is supplied to each basin by four 25-hp, two-speed, floating mechanical aerators. The maximum water depth is 12 feet. The design was based on an organic loading of 0.23 lb BOD/lb MLVSS (26 lb BOD/1000 C.F.) and an aeration time of 24 hours at design BOD loading.

TABLE 2
DESIGN FACTORS

INFLUENT PUMPS	
Number	2
Type	Centrifugal
Capacity	2400 gpm
Total Head	26 feet
HEADWORKS	
Sewage Grinder	
Number	1
Size	25-inch
AERATION BASINS	
Number	2
Depth	12 feet
Volume (each)	1.0 million gallons
Hydraulic Capacity (each)	6.0 mgd
Design Organic Loadings[1]	0.23 lb BOD/lb MLVSS/day (26 lb BOD/1000 cu. ft./day)
AERATION EQUIPMENT	
Number of Aerators (each basin)	4
Type	2-speed, floating, mechanical surface type
Size	25-hp
CLARIFIER	
Number	1
Diameter	60 feet
Side Water Depth	10 feet
Surface Overflow Rate[2]	707 gal/d/sq. ft.
Solids Loading Rate[2,3]	26 lb/d/sq. ft.
Detention Time[2]	2.5 hours
Sludge Removal	Revolving suction arm
SLUDGE RECIRCULATION	
Number of Pumps	2
Type	1 single-speed 1 variable-speed
Recirculation Ratio[2]	0.25 to 1.0

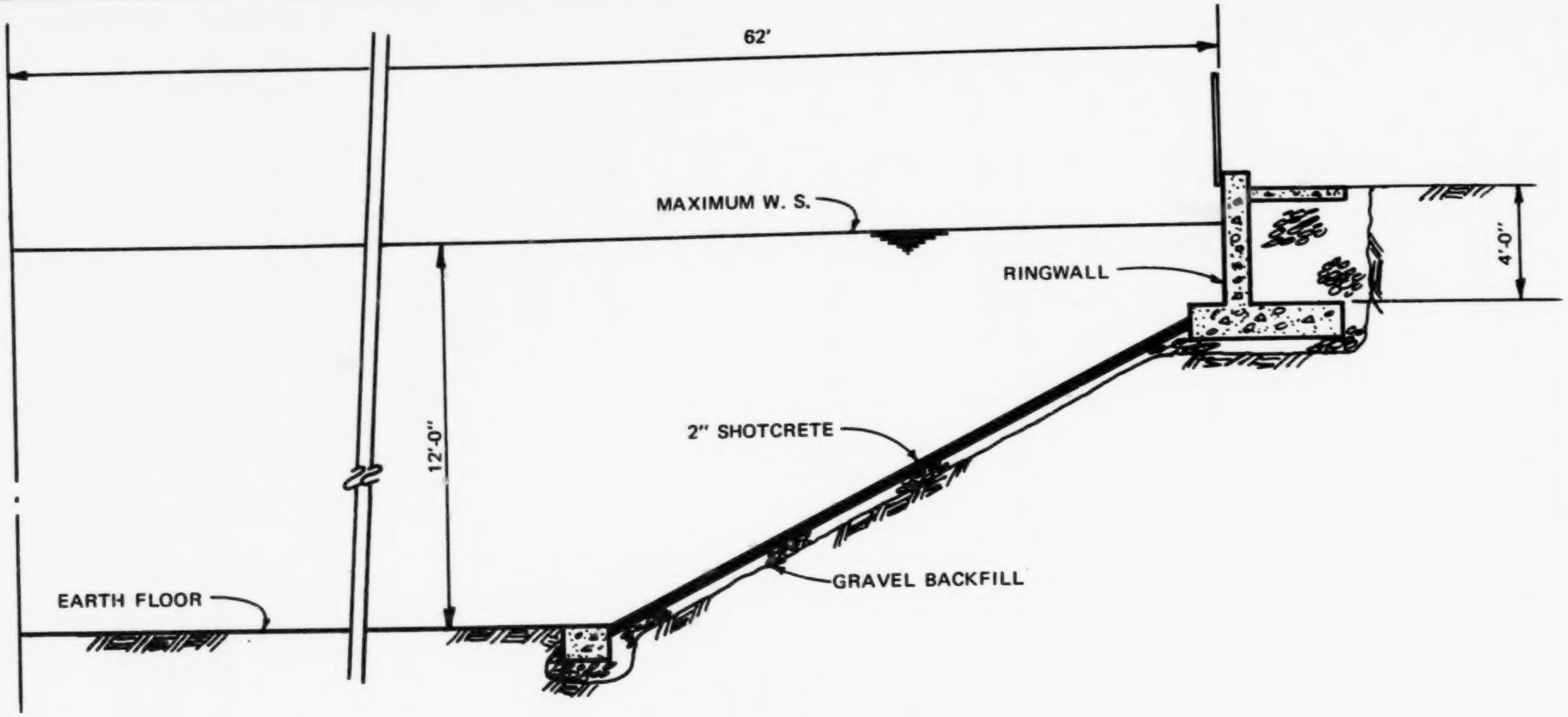
FLOW MEASUREMENT	
Plant Flow	Prop. meter, clarifier discharge
Recycled Sludge Flow	Prop. meter behind pumps
Waste Sludge	Time clocks, constant pump rate
CHLORINATION	
Type	V-notch chlorinator
Control	Plant flow
CHLORINE CONTACT	
Unit	Outfall and chlorine contact channel
Detention Time[2]	1.55 hours
SLUDGE DIGESTION	
Number	1
Type	Aerobic flow-through without thickening
Volume	480,000 gallons
Detention Time[4]	
4000 mg/l solids to digester	5.2 days
8000 mg/l solids to digester	10.4 days
Oxygen Supply and Mixing	30-hp floating mechanical aerator
Minimum Digested Sludge Age[4]	
4000 mg/l solids to digester	15.2 days
8000 mg/l solids to digester	20.4 days
DIGESTED SLUDGE DISPOSAL	
Method	Humus ponds
Number	2
Area, each	67,000 sq. ft.
Total	3.1 acres
Supernatant Drainage	Return to plant
Operation	Dry to 3-foot depth of water
EFFLUENT POLISHING	
Method	Discharge to humus pond No. 1
Detention Time[2]	20 hours

[1] At 3500 lb. BOD/day/basin and 2200 mg/l MLSS

[2] At average design flow of 2.0 mgd

[3] At 2200 mg/l MLSS and 1.0 recirculation ratio

[4] At design loading


62'
MAXIMUM W. S.
RINGWALL
4'-0"
12'-0"
2" SHOTCRETE
GRAVEL BACKFILL
EARTH FLOOR

NOTES: BASIN SYMMETRICAL ABOUT CENTER LINE. CONCRETE PADS PLACED UNDER EACH AERATOR

FIGURE 3

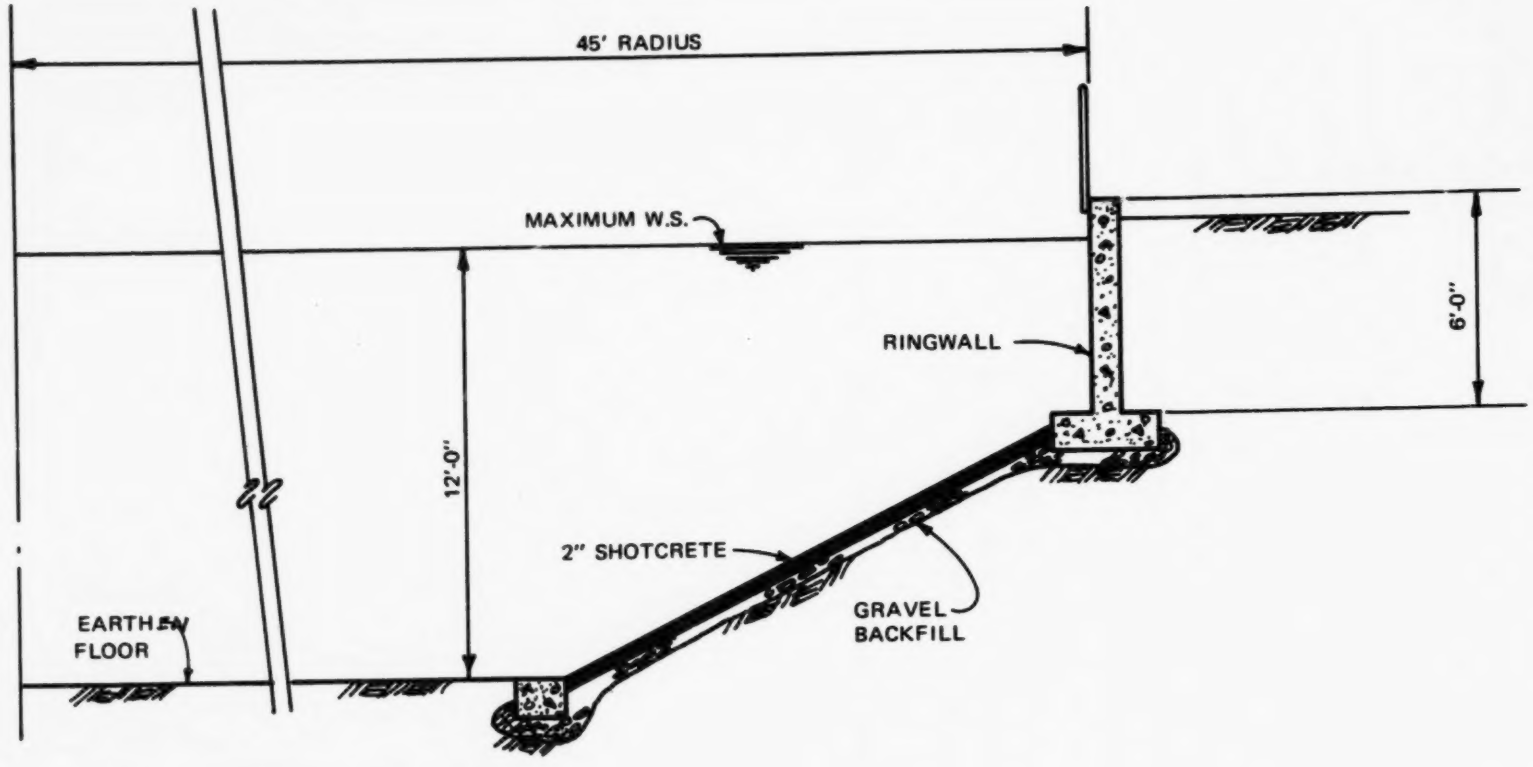
AERATION BASIN
TYPICAL SECTION

The 60-foot diameter clarifier has a 10-foot sidewater depth. Recycled activated sludge is removed by a rapid sludge withdrawal mechanism utilizing suction pipes mounted on the scraper arms. Waste sludge is scraped to the center of the unit and removed through a sludge hopper. The design was based on a hydraulic overflow rate of 707 gal/day/sq. ft. and a solids loading of 26 lb/day/sq. ft. at a mixed liquor flow of 4.0 mgd and a mixed liquor suspended solids (MLSS) level of 2,200 mg/l.

The maximum capacity of the aerobic digester is 480,000 gallons. The basin is 90 feet in diameter with a maximum water depth of 12 feet. The digester construction is similar to that of the aeration basins, i.e., earth basin with a reinforced concrete ring wall and shotcrete lined sideslopes. Figure 4 is a typical section through the aerobic digester. Oxygen is supplied to the basin by a single-speed 30-hp floating mechanical aerator. The design was based on a hydraulic detention time, at design loading, of 10.4 days with a waste sludge solids concentration of 8000 mg/l.

Chlorine contact time is provided in a shotcrete lined channel, 450 feet in length. The design provides a 1.55 hour contact time at average design flow. A typical section through the chlorine contact channel is shown on Figure 5.

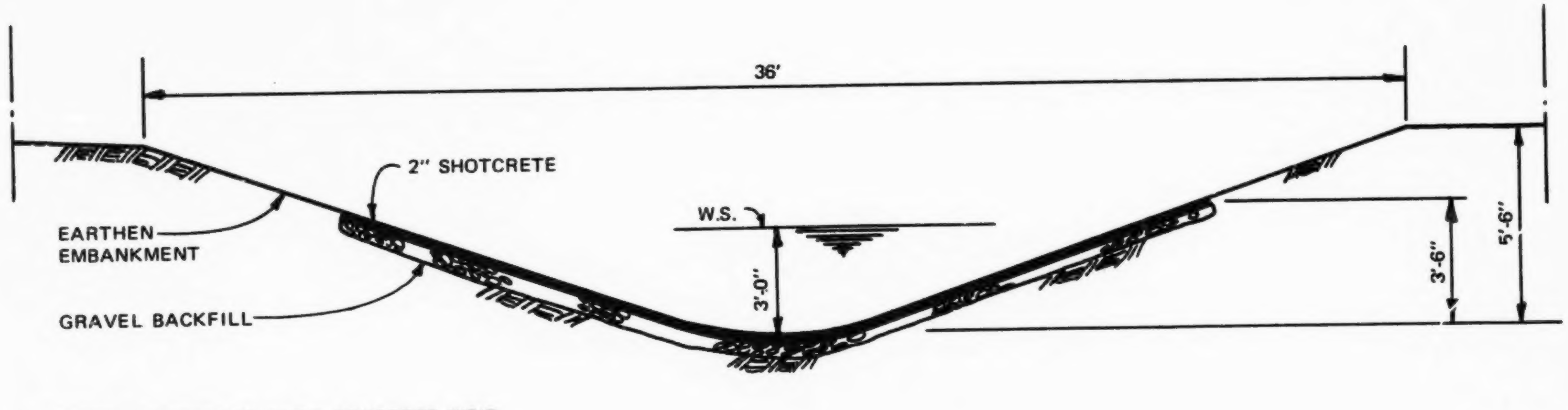
The control building houses the plant motor control center, automatic samplers, laboratory, and waste sludge pumps.


45' RADIUS
MAXIMUM W.S.
RINGWALL
6'-0"
12'-0"
2" SHOTCRETE
GRAVEL BACKFILL
EARTHEN FLOOR

NOTES: BASIN SYMMETRICAL ABOUT CENTER LINE. CONCRETE PAD PLACED UNDER AERATOR

FIGURE 4

AEROBIC DIGESTER
TYPICAL SECTION


36'
2" SHOTCRETE
W.S.
EARTHEN EMBANKMENT
GRAVEL BACKFILL
3'-0"
3'-6"
5'-6"

NOTF: DISCHARGE OVER CONCRETE WEIR

FIGURE 5

CHLORINE CONTACT CHANNEL
TYPICAL SECTION

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SECTION V
DEMONSTRATION PROCEDURES

PLANT STARTUP

Initial aeration basin startup was accomplished as described below.

Prior to startup, the aeration basins were partially filled with water, to test the aerator operation. Aeration basin number 2 was seeded with 8,000 gallons of secondary sludge from the City's old trickling filter plant, and approximately one cubic yard of barnyard manure obtained from an adjacent farm. All four aerators were run at low speed to mix the basin contents and increase the dissolved oxygen level. Raw wastewater flow into the basin was begun after one day of aeration and was gradually increased to 0.75 mgd over a period of about one week. Three days were required to fill the basin because of the low initial flow. All biological solids produced were returned to the aeration basin until the MLSS reached the desired level.

Acclimation of the system was monitored with daily COD determinations on both influent and effluent flows and suspended solids analyses of the aeration basin contents. The system was assumed to be completely acclimated after approximately four weeks of operation when the daily COD removals leveled off above 90 percent.

Aeration basin number 1 was started after basin number 2 became acclimated. All of the activated sludge was returned to basin number 1 for 36 hours prior to the addition of any wastewater. The entire wastewater flow was diverted to the basin at the end of this period.

OPERATION

The treatment system was operated continuously from August 1969, through November 1970. Table 3 is a detailed schedule of operations during the demonstration period.

SAMPLING SCHEDULE AND PROCEDURES

Samples were taken at various locations throughout the system. These sample point locations are shown on Figure 2, and are described below. All subsequent discussion of sample points will refer to Figure 2. Table 4 lists the sampling and testing schedules used throughout the demonstration period.

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TABLE 3

DETAILED OPERATION SCHEDULE

DATE	CANNERY(1) OPERATION	AERATION BASINS USED	AVERAGE PLANT FLOW (mgd)	AVERAGE(2) AERATION TIME (hrs.)	AVERAGE MLSS(3) (mg/l)	AEROBIC DIGESTER DETENTION TIME (days)	DATE
AUGUST 1969	GREEN BEANS		START-UP & ACCLIMATION				AUGUST 1969
SEPTEMBER	PRUNES				2900		SEPTEMBER
OCTOBER			1.2	20	1400		OCTOBER
NOVEMBER						40	NOVEMBER
DEC. 1969		ONE			1700		DEC. 1969
JAN. 1970			4.3	5.6		80	JAN. 1970
FEBRUARY							FEBRUARY
MARCH					1300	55	MARCH
APRIL			1.6	15			APRIL
MAY						90	MAY
JUNE		TWO		50	1500	35	JUNE
JULY					1300	100	JULY
AUGUST	GREEN BEANS		0.95	25	2000	70	AUGUST
SEPTEMBER	PRUNES	ONE					SEPTEMBER
OCTOBER					1000	50	OCTOBER
NOV. 1970			2.0	12		130	NOV. 1970

(1) CANNERY OPERATION BASED ON ACTUAL RECORDS FOR 1969 AND 1970 ADDITIONAL PRODUCTS MAY BE PRODUCED IN THE FUTURE.

(2) AERATION TIME WAS SET BY THE NUMBER OF AERATION BASINS IN OPERATION AND THE INFLUENT FLOW RATE.

(3) MIXED LIQUOR SUSPENDED SOLIDS.

TABLE 4

SAMPLING AND TESTING SCHEDULES

(NUMBER OF TESTS PER WEEK)

CANNING SEASON (JULY 15 TO SEPT. 30)

	SAMPLE POINT	BOD TOTAL	BOD DISSOLVED	SUSPENDED SOLIDS			pH	TEMPERATURE	DISSOLVED OXYGEN	COLIFORM	NITROGEN(1)	PHOSPHOROUS(2)	ALKALINITY	CHLORINE RESIDUAL	MICROSCOPIC EXAM
				TOTAL	VOLATILE	SETTLEABLE									
A	PLANT INFLUENT	3	–	3	3	5	5	10	5	–	1	1	3	–	–
B	AERATION BASIN EFFLUENT	–	–	3	3	5	5	10	5	–	–	–	3	–	1
C	WASTE ACTIVATED SLUDGE	–	–	3	3	–	–	–	–	–	–	–	–	–	–
C	AEROBIC DIGESTER	–	–	3	3	–	5	10	5	–	–	–	3	–	1
E	PLANT EFFLUENT	3	3	3	3	5	5	5	5	1	1	1	3	5	–
F	POLISHING POND EFFLUENT	•	–	•	–	–	–	–	–	–	–	–	–	–	–

NON-CANNING SEASON (OCT. 1 TO JULY 14)

	SAMPLE POINT	BOD TOTAL	BOD DISSOLVED	SUSPENDED SOLIDS			pH	TEMPERATURE	DISSOLVED OXYGEN	COLIFORM	NITROGEN (1)	PHOSPHOROUS(2)	ALKALINITY	CHLORINE RESIDUAL	MICROSCOPIC EXAM
				TOTAL	VOLATILE	SETTLEABLE									
A	PLANT INFLUENT	2	–	2	2	3	3	5	5	Bi-Weekly	Tri-Weekly	Tri-Weekly	2	–	–
B	AERATION BASIN EFFLUENT	–	–	2	2	3	3	5	5	–	–	–	2	–	1
C	WASTE ACTIVATED SLUDGE	–	–	2	2	–	–	–	–	–	–	–	–	–	–
D	AEROBIC DIGESTER	–	–	2	2	–	3	5	5	–	–	–	2	–	1
E	PLANT EFFLUENT	2	1	2	2	3	3	5	5	Bi-Weekly	Tri-Weekly	Tri-Weekly	2	5	–
F	POLISHING POND EFFLUENT	•	–	•	–	–	–	–	–	–	–	–	–	–	–

(1) TOTAL KJELDAHL, AMMONIA AND NITRATE
(2) TOTAL AND ORTHOPHOSPHATE
• SAMPLING FREQUENCY AS REQUIRED

PLANT INFLUENT–Automatic 24-hour composite samples, proportional to plant flow, were obtained by pumping from the headworks, downstream of the comminutor. This location is shown as sample point A on Figure 2. The automatic sampler, sample pumps and storage refrigerator are located in the control building.

AERATION BASIN EFFLUENT–These samples were obtained by manually compositing grab samples during the daily 8-hour operating period. The location is shown as sample point B.

WASTE ACTIVATED SLUDGE–Grab samples taken at the waste sludge pump were mixed to form a composite sample during the operating period. The location is shown as sample point C.

AEROBIC DIGESTER–Sample point D shows the location of these samples, which were manually mixed to form composites from grab samples taken from the digester at the outlet box.

PLANT EFFLUENT–Automatic 24-hour composite samples, proportional to plant flow, were obtained by pumping from the clarifier effluent line at the flow measurement box, upstream from the point of chlorine injection. The sampler and pump are located in the control building adjacent to those for the plant influent. This sample location is shown as sample point E.

POLISHING POND EFFLUENT–Samples of the polishing pond effluent (sample point F) were not taken on a regular schedule after acclimation of the system since, after this period, the polishing pond was used only for effluent bypass while the chlorine contact channel was cleaned. The majority of the samples were grab samples taken at the entrance to the outfall pipe.

ANALYTICAL METHODS

All analyses, with the exception of nitrogen, phosphorous and coliform tests, were performed in the plant laboratory. Nitrogen, phosphorous and coliform analyses were performed at the EPA's Pacific Northwest Water Laboratory in Corvallis, Oregon.

All plant testing was done in accordance with the twelfth edition (1965) of "Standard Methods for the Examination of Water and Wastewater" of the American Public Health Association [11], with the following exceptions:

SUSPENDED SOLIDS–Suspended solids tests were performed using Whatman No. 541 hardened ashless filter discs instead of asbestos mats.

Settleable solids were measured by both the volume method and the weight method. Use of the weight method was discontinued early in the program because of the very small quantity of settleable solids encountered.

Nitrogen and phosphorous tests were performed in accordance with the FWPCA manual [12].

A list of laboratory equipment used for the analytical testing is included in Appendix B.

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SECTION VI
WASTEWATER CHARACTERISTICS

GENERAL

Influent wastewater data was separated into three different categories to obtain a more meaningful analysis. The three categories are as follows: (1) canning season, (2) dry weather non-canning season, and (3) wet weather non-canning season.

The canning season normally extends from about 15 July through 30 September, and the wet weather non-canning season from 10 December through 20 March. The dry weather non-canning season occurs in two separate periods: 1 October through 9 December, and 21 March through 14 July.

Table 5 lists average, maximum, minimum and standard deviation data for the various parameters used to evaluate the influent wastewater characteristics. These values were calculated from data for each category described previously. These data are discussed in the following paragraphs.

CANNING SEASON

ALKALINITY AND pH–The influent wastewater alkalinity during the canning season varied from 68 mg/l to 129 mg/l with an average value of 99 mg/l, as $CaCO_3$. The pH varied from 6.0 to 7.4 and averaged 7.0.

TEMPERATURE AND DISSOLVED OXYGEN (D.O.)–The temperature ranged from 14 degrees C to 22 degrees C and averaged 20 degrees C. The minimum D.O. level was 0.3 mg/l, the maximum level was 1.7 mg/l, and the average was 0.5 mg/l.

FLOW–The wastewater flow averaged about 1.0 mgd during this period and varied from a minimum of 0.4 mgd to a maximum of 2.4 mgd. Figure 6 shows influent flow vs. time for the demonstration period.

BIOCHEMICAL OXYGEN DEMAND (BOD)–The influent BOD averaged 166 mg/l (1108 lb/day) and ranged from 59 mg/l (497 lb/day) to 635 mg/l (3813 lb/day). Figure 7 shows influent BOD in mg/l vs. time and Figure 8 shows influent BOD in lb/day vs. time for the total operating period.

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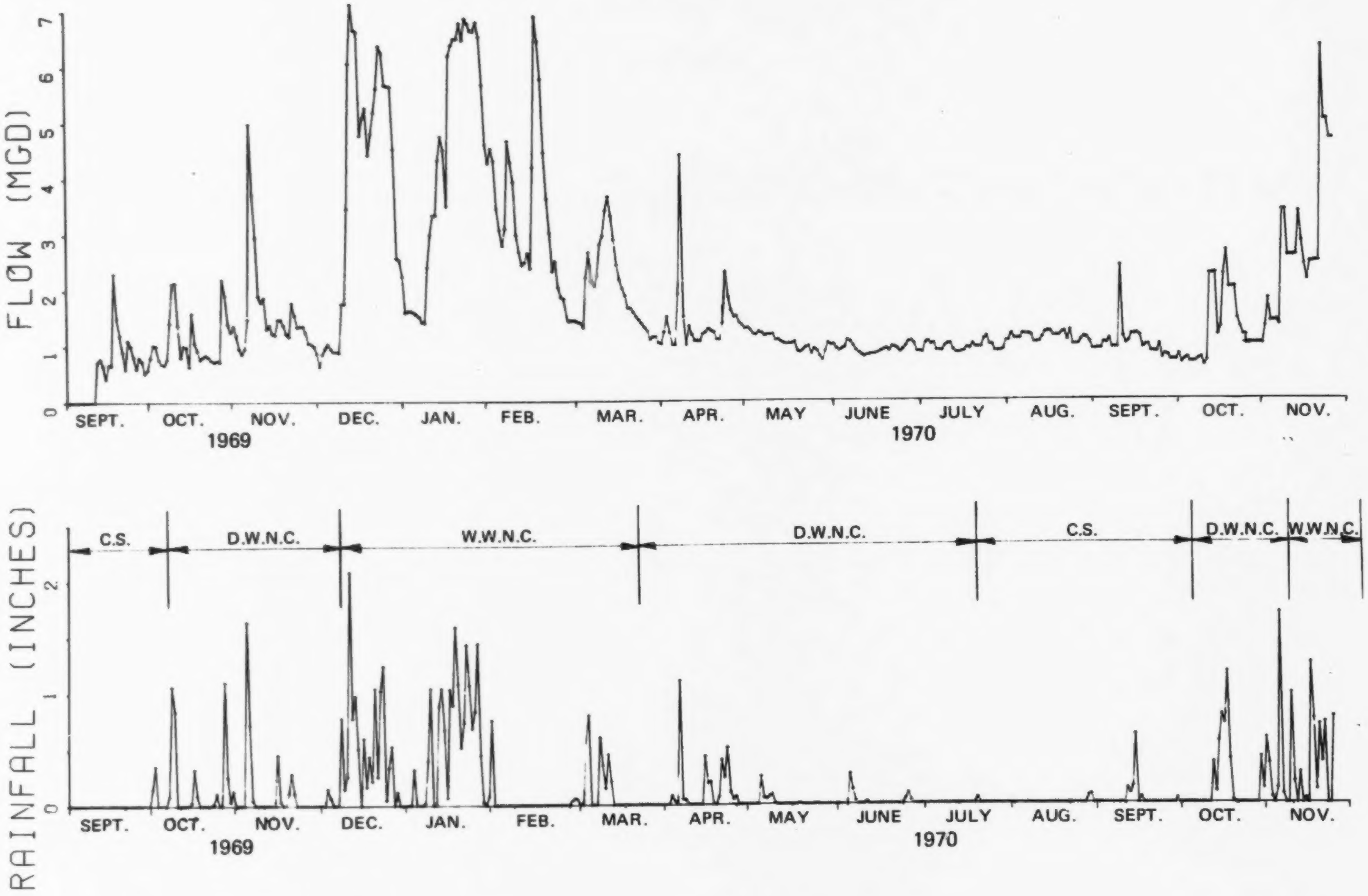
TABLE 5

INFLUENT CHARACTERISTICS

	* * * * CANNING SEASON * * * *				* * * * DRY WEATHER NON-CANNING * * * *				* * * * WET WEATHER NON-CANNING * * * *			
	AVG.	MAX.	MIN.	STD. DEV.	AVG.	MAX.	MIN.	STD. DEV.	AVG.	MAX.	MIN.	STD. DEV.
FLOW (MGD)	0.98	2.39	0.42	0.27	1.18	4.97	0.60	0.55	3.66	7.11	0.87	1.76
BOD (MG/L)	165.8	635.0	59.0	126.7	95.3	195.0	35.0	31.2	40.4	123.0	6.0	26.2
BOD (LB/D)	1107.7	3813.0	496.9	624.6	880.5	2353.8	304.7	338.1	1023.9	2988.0	222.1	613.4
TSS (MG/L)	124.3	552.0	28.0	110.3	94.1	320.0	8.0	62.7	65.2	310.0	12.0	59.8
TSS (LB/D)	908.7	3314.6	233.5	637.1	966.5	5631.1	98.0	942.9	1891.1	13107.9	458.6	2401.7
VSS (MG/L)	62.7	232.0	14.0	48.5	76.5	320.0	1.0	63.8	48.9	260.0	2.0	52.2
VSS (LB/D)	577.5	2109.0	113.2	492.7	798.2	5261.6	8.9	972.2	1429.6	10993.7	78.9	2022.2
ALK (MG/L)	98.5	129.0	68.0	14.4	97.0	132.0	48.0	18.7	56.8	91.0	37.0	12.5
PH	7.0	7.4	6.0	0.3	7.2	7.6	6.8	0.1	7.1	7.5	6.7	0.1
TEMP (DEG C)	19.6	22.0	14.0	1.2	15.7	21.0	8.0	2.3	11.5	16.0	9.0	1.7
D.O. (MG/L)	0.5	1.7	0.3	0.2	1.3	6.0	0.3	1.0	4.5	7.1	1.6	1.9
TKN (MG/L)	14.4	35.5	1.4	11.5	10.0	22.9	0.1	7.4	7.7	13.3	0.4	6.5
NH3-N (MG/L)	4.9	11.7	0.1	3.8	5.7	10.0	0.8	3.2	4.3	5.6	3.3	1.1
NO3-N (MG/L)	1.0	9.6	0.0	3.0	1.6	7.2	0.1	2.8	1.1	2.3	0.2	1.0
T-PO4 (MG/L)	5.4	7.2	3.9	1.1	5.2	7.2	1.9	1.5	2.4	3.9	0.9	1.4
O-PO4 (MG/L)	3.2	5.8	1.3	1.4	3.6	6.9	1.6	1.6	1.4	2.7	0.3	1.2



C.S. = CANNING SEASON
D.W.N.C. = DRY WEATHER NON-CANNING
W.W.N.C. = WET WEATHER NON-CANNING

FIGURE 6

FLOW AND RAINFALL VS. TIME

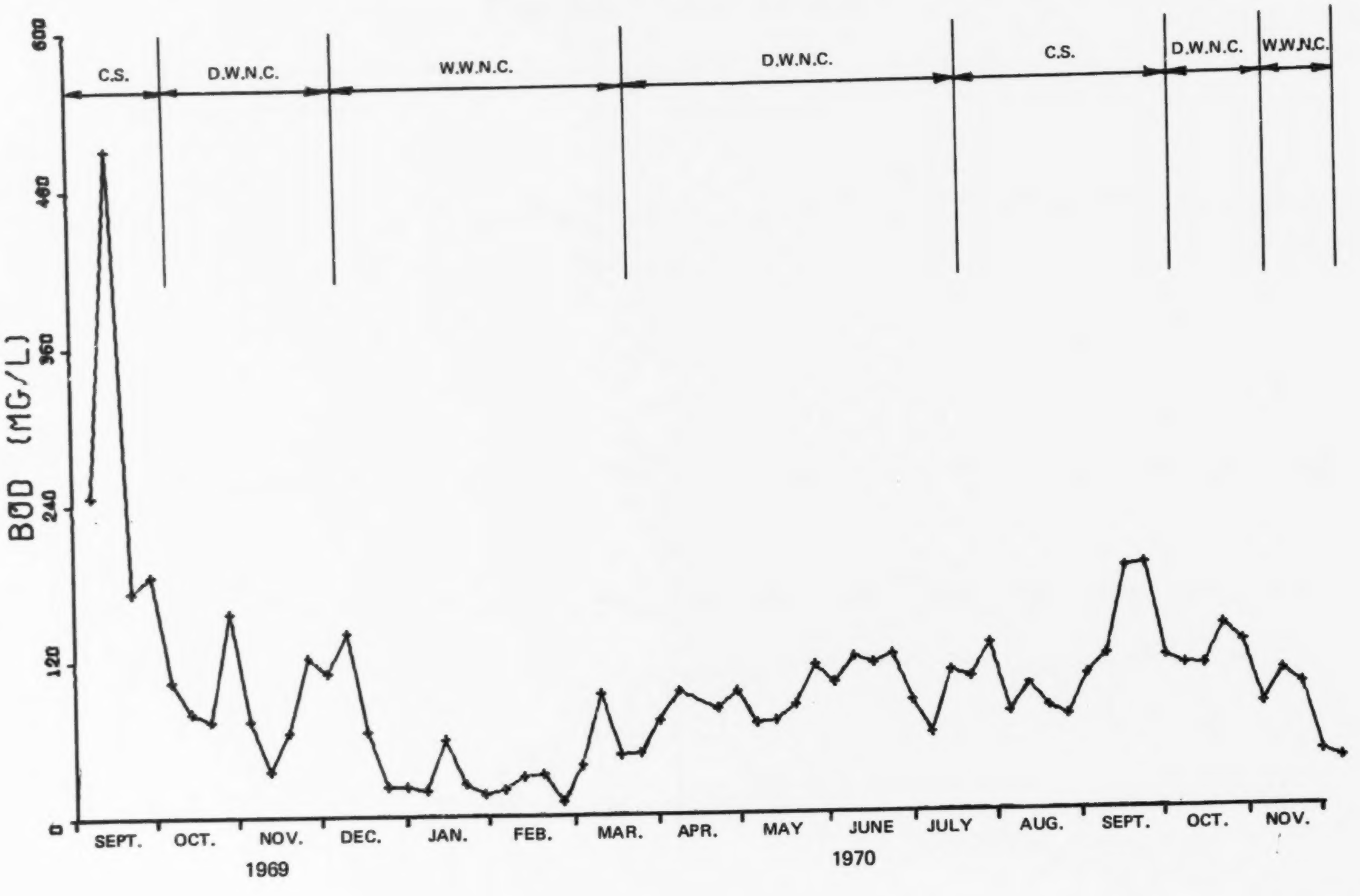


FIGURE 7

INFLUENT BOD VS. TIME
WEEKLY AVERAGE DATA

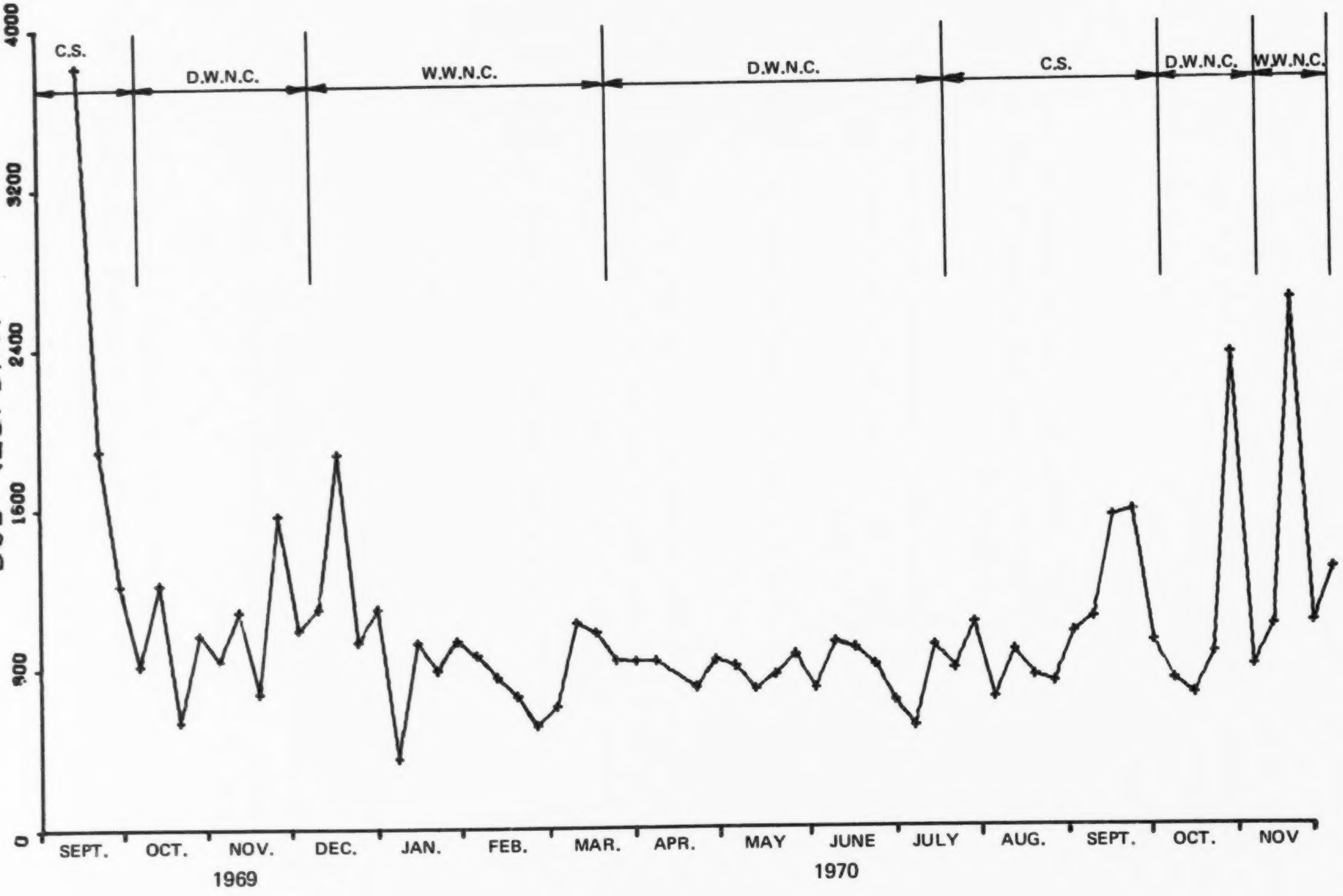


FIGURE 8

BOD LOADING (LB./DAY) VS. TIME
WEEKLY AVERAGE DATA

TOTAL AND VOLATILE SUSPENDED SOLIDS (TSS, VSS)–The average TSS was 124 mg/l of which 63 mg/l, or 51 percent, was VSS. The range was from a low TSS value of 28 mg/l (14 mg/l VSS) to a high of 552 mg/l (232 mg/l VSS).

NUTRIENTS–The BOD:nitrogen:phosphate ratio in the influent wastewater averaged 100:8.7:3.3. The average ammonia nitrogen concentration was 4.9 mg/l, and the total phosphate concentration averaged 5.4 mg/l.

DRY WEATHER NON-CANNING SEASON

ALKALINITY AND pH–The influent wastewater alkalinity during the dry weather non-canning season varied from 48 mg/l to 132 mg/l and averaged 97 mg/l, as $CaCO_3$. The pH ranged from 6.8 to 7.6 and averaged 7.2.

TEMPERATURE AND D.O.–The average influent temperature during this period was 16 degrees C with a range from 8 degrees C to 21 degrees C. The D.O. level ranged from 0.3 mg/l to 6.0 mg/l with an average of 1.3 mg/l.

FLOW–The influent flow varied from 0.6 mgd to 5 mgd, as shown by Figure 6, and averaged 1.2 mgd for this period.

BOD–The average influent BOD was 95 mg/l with a range from 35 mg/l to 195 mg/l. BOD to the plant, in pounds per day, averaged 880 and varied from 305 lb/day to 2550 lb/day. These relationships are shown on Figures 7 and 8.

TSS AND VSS–The average TSS concentration was 94 mg/l of which 76.5 mg/l or 81 percent was volatile.

NUTRIENTS–The average BOD:nitrogen:phosphate ratio was 100:10:5.5.

WET WEATHER NON-CANNING SEASON

ALKALINITY AND pH–The alkalinity ranged from 37 mg/l to 91 mg/l and averaged 57 mg/l, as $CaCO_3$, during the wet weather non-canning season. The pH range was from 6.7 to 7.5 and the average pH was 7.1.

TEMPERATURE AND D.O.–The temperature of the influent wastewater averaged 11.5 degrees C and varied from 9 degrees C to 16 degrees C. D.O. levels ranged from 1.6 mg/l to 7.1 mg/l and averaged 4.5 mg/l.

FLOW–Referring again to Figure 6, the influent flow varied from 0.9 mgd to 7.1 mgd, and averaged 3.7 mgd.

BOD–The average BOD during this period was 40 mg/l (1024 lb/day) and ranged from 6 mg/l (222 lb/day) to 123 mg/l (2988 lb/day). These relationships are also shown on Figures 7 and 8.

TSS AND VSS–TSS averaged 65 mg/l and varied from 12 mg/l to 310 mg/l. VSS averaged 49 mg/l and varied from 2 mg/l to 260 mg/l.

NUTRIENTS–The average BOD:nitrogen:phosphate ratio during the wet weather non-canning season was 100:19:6.

INFILTRATION

The flow data discussed above shows a wide variation in the quantity of wastewater received at the plant. Figure 6 shows the correlation between plant flow and rainfall data. In general, during the wet weather, the peak flows are associated with rainy periods. This phenomenon is primarily due to infiltration of groundwater into the sewer system. This effect is much less prevalent in the summer and fall before the ground has become saturated.

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SECTION VII
TREATMENT PLANT PERFORMANCE

GENERAL

The plant was scheduled for startup prior to the beginning of the 1969 canning season. However, due to construction delays, startup was not accomplished until near the peak canning period in August. Following startup, the plant was operated continuously through November 1970, including the full 1970 canning season.

EFFLUENT QUALITY AND SYSTEM STABILITY

Influent and effluent wastewater quality parameters were monitored throughout the operating period to provide a basis for analysis of the system stability and performance. These data were analyzed in conjunction with other variables which affect the operation of biological systems.

EFFLUENT QUALITY–Figure 9 is a plot of weekly average influent and effluent BOD, in mg/l, vs. time, for the entire operating period. Figure 10 is a similar plot for influent and effluent total suspended solids data. These figures show a consistently good effluent quality throughout the year, regardless of the influent loading, flow, or operating temperature encountered.

A summary of effluent data is listed in Table 6. This summary is separated into canning season, dry weather non-canning season, and wet weather non-canning season. It includes average, maximum, minimum and standard deviation values for all effluent data for each period. A summary of influent and effluent average, maximum, minimum and standard deviation data for the entire demonstration period is listed in Table 7.

During the canning season, the effluent BOD averaged 7.8 mg/l with a maximum value of 16 mg/l. The average effluent TSS was 11.8 mg/l and ranged from 1 mg/l to 31 mg/l.

The average BOD and TSS values during the dry weather non-canning period were, respectively, 8.4 mg/l and 13.9 mg/l.

The averages during the wet weather non-canning season were 7.3 mg/l BOD and 13.9 mg/l TSS.

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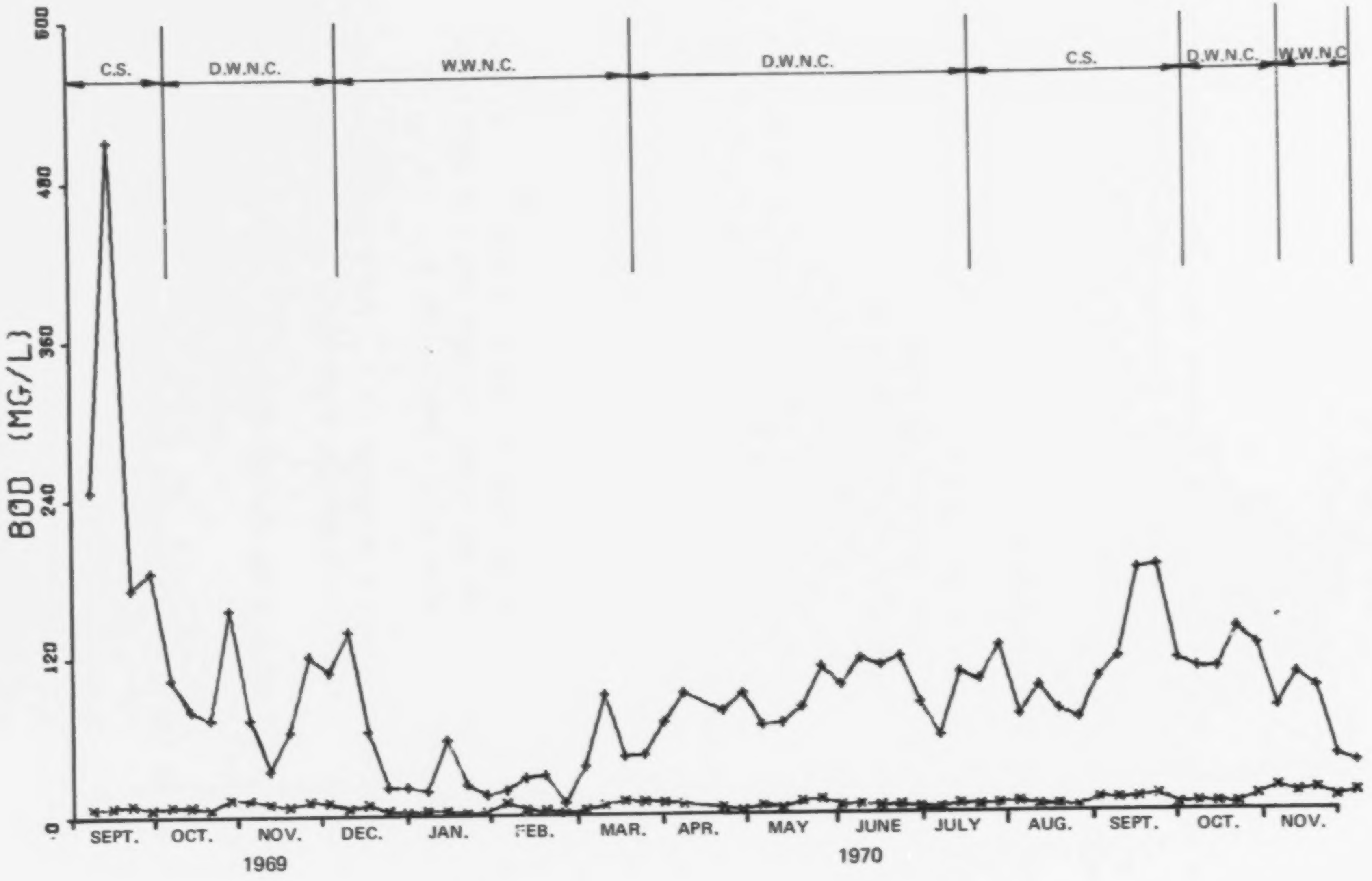


FIGURE 9

INFLUENT AND EFFLUENT BOD VS. TIME
WEEKLY AVERAGE DATA

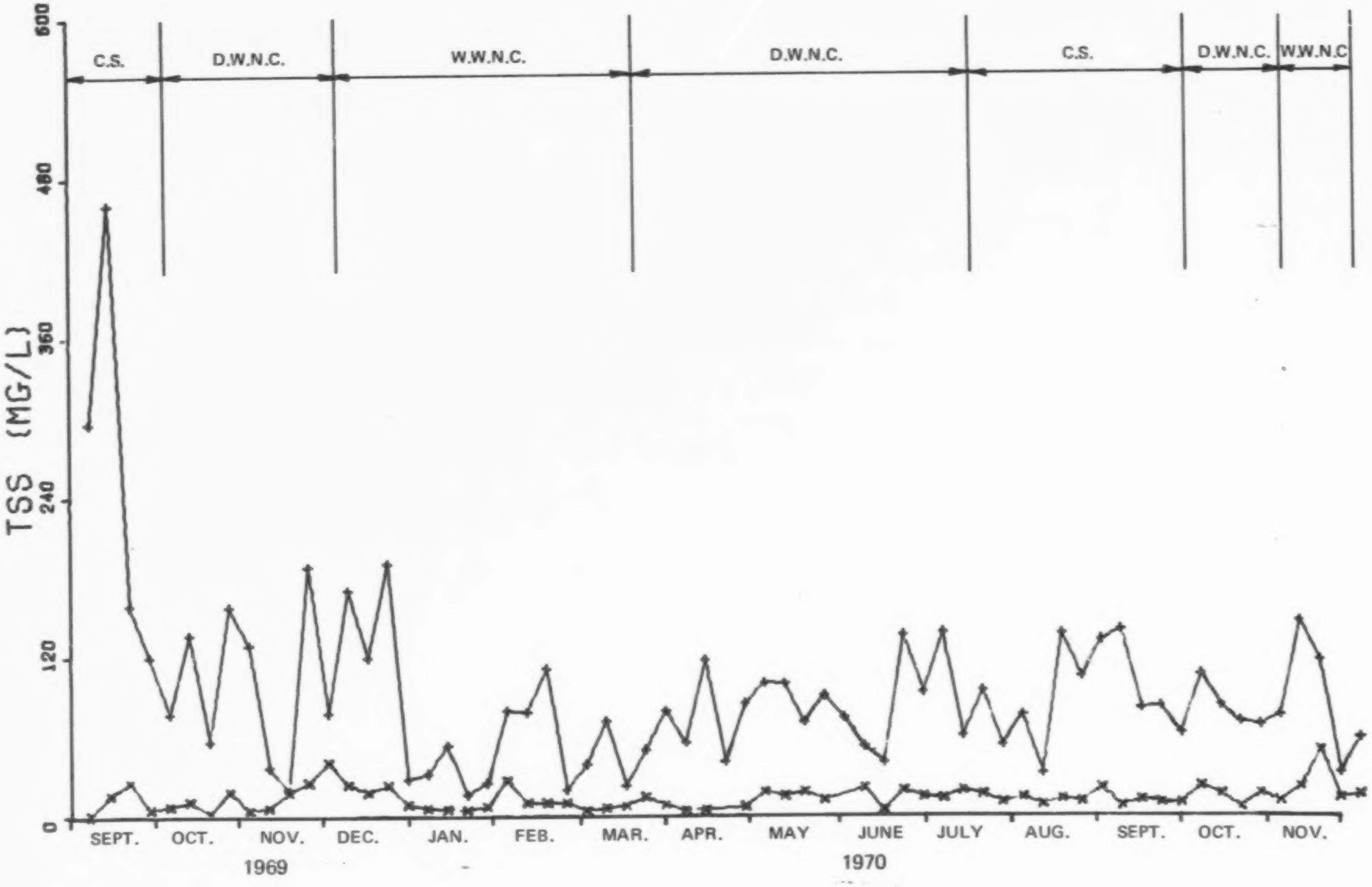


FIGURE 10

INFLUENT AND EFFLUENT TSS VS. TIME
WEEKLY AVERAGE DATA

TABLE 6

EFFLUENT CHARACTERISTICS

	* * * * CANNING SEASON * * * *				* * * * DRY WEATHER NON-CANNING * * * *				* * * * WET WEATHER NON-CANNING * * * *			
	AVG.	MAX.	MIN.	STD. DEV.	AVG.	MAX.	MIN.	STD. DEV.	AVG.	MAX.	MIN.	STD. DEV.
BOD (MG/L)	7.76	16.00	3.00	2.93	8.42	22.00	3.00	4.05	7.32	20.00	2.00	5.16
BOD (LB/D)	66.8	134.2	25.0	28.6	83.9	224.1	23.5	51.8	211.2	835.6	23.5	195.9
SOL BOD (MGL)	2.4	6.0	1.0	1.4	3.5	9.0	1.0	2.9	2.6	5.0	1.0	1.3
TSS (MG/L)	11.8	31.0	1.0	7.8	13.9	40.0	1.0	9.2	13.9	68.0	1.0	13.3
TSS (LB/D)	109.4	498.7	6.7	95.1	138.3	368.6	8.7	99.2	451.0	1916.8	16.9	468.2
VSS (MG/L)	7.6	25.0	1.0	7.4	12.3	48.0	1.0	10.2	11.8	42.0	1.0	10.8
VSS (LB/D)	73.7	326.0	6.7	83.7	122.2	540.4	6.6	112.5	377.8	1607.7	11.9	376.9
ALK (MG/L)	62.8	93.0	27.0	18.4	34.1	65.0	7.2	11.6	38.7	66.0	23.0	8.1
PH	7.1	7.5	6.5	0.2	7.0	7.4	6.6	0.1	7.1	7.4	6.5	0.1
TEMP (DEG C)	19.6	22.0	16.0	1.4	17.5	21.0	7.2	2.5	14.6	16.0	13.0	0.8
D.O. (MG/L)	5.0	8.6	3.2	1.3	6.0	10.0	3.5	1.1	6.5	8.9	4.4	1.0
TKN (MG/L)	2.3	10.5	0.2	2.8	3.3	12.3	1.1	3.2	1.1	1.6	0.8	0.3
NH3-N (MG/L)	0.5	5.6	0.0	1.6	0.8	5.0	0.0	1.5	0.2	0.6	0.0	0.2
NO3-N (MG/L)	3.5	16.4	0.4	4.7	5.9	10.2	0.3	3.2	5.1	7.0	3.1	1.7
T-PO4 (MG/L)	4.4	6.7	1.1	1.5	5.3	7.8	2.6	1.5	2.1	3.4	0.5	1.4
O-PO4 (MG/L)	4.2	6.1	3.0	1.0	4.3	6.4	2.3	1.1	1.8	3.2	0.3	1.3

TABLE 7

INFLUENT AND EFFLUENT CHARACTERISTICS

- ALL DATA -

CHARACTERISTIC	* * * * * INFLUENT * * * * * AVERAGE	MAXIMUM	MINIMUM	STANDARD DEVIATION	* * * * * EFFLUENT * * * * * AVERAGE	MAXIMUM	MINIMUM	STANDARD DEVIATICN
FLOW (MGD)	1.84	7.11	0.42	1.54	--	--	--	--
BOD (MG/L)	103.9	635.0	6.0	88.9	7.9	22.0	2.0	4.0
BOD (LB/D)	983.5	3,813.0	222.1	517.0	112.3	835.6	23.5	121.6
SOL BOD (MGL)	--	--	--	--	2.9	16.0	1.0	2.1
TSS (MG/L)	96.6	552.0	8.0	82.7	13.2	68.0	1.0	10.1
TSS (LB/D)	1,178.5	13,107.9	98.0	1,444.0	218.2	1,916.8	6.7	298.7
VSS (MG/L)	65.3	320.0	1.0	57.7	10.9	48.0	1.0	9.8
VSS (LB/D)	912.2	10,993.7	8.9	1,301.3	179.7	1,607.7	6.6	247.8
ALK (MG/L)	85.8	132.0	37.0	24.4	42.5	93.0	7.2	17.3
PH	7.1	7.6	6.0	0.2	7.0	7.5	6.5	0.1
TEMP (DEG C)	15.7	22.0	8.0	3.4	18.2	22.0	7.2	2.5
D.O. (MG/L)	1.3	7.1	0.3	1.5	5.9	10.0	3.2	1.2
TKN (MG/L)	11.7	35.5	0.1	9.4	2.5	12.3	0.2	2.8
NH3-N (MG/L)	5.1	11.7	0.1	3.2	0.6	5.6	--	1.4
NO3-N (MG/L)	1.3	9.6	--	2.6	4.8	16.4	0.3	3.8
T-PO4 (MG/L)	4.9	7.2	0.9	1.6	4.3	7.8	0.5	1.8
O-PO4 (MG/L)	3.1	6.9	0.3	1.6	3.9	6.4	0.3	1.4

CANNERY STARTUP, OPERATION AND SHUTDOWN–Figure 11 is a plot of influent and effluent BOD and effluent soluble BOD vs. time, during the canning season.

The plant was started up during the 1969 canning season. The peak BOD loads shown on Figure 11 for the period from 9 September through 12 September, however, were not attributable to waste from the cannery. During this period, a high concentration of plywood glue waste appeared in the plant influent. There was, however, no significant deterioration of effluent quality as a result of these peak loads.

The peak BOD load during the 1970 canning season was significantly less than for the 1969 season, even after eliminating the peaks associated with the glue waste. This can probably be attributed to a poor crop yield in 1970. The peak period again occurred during September with no significant loss in effluent quality.

There appeared to be no significant effect on effluent quality as a result of cannery startup or shutdown. This could be partially due to a gradual increase and decrease in processing by the cannery.

Figure 12 shows percentage of BOD removal during the same period of time. BOD removal during the canning season was greater than 90 percent most of the time. The efficiency dropped to about 88 percent following the 1969 cannery shutdown. However, a comparison with Figure 11 shows that only a small increase in effluent BOD was observed and the reduced efficiency was principally due to the reduction in influent BOD concentration.

The drop in efficiency to about 86 percent BOD removal following cannery startup in 1970 was due only to a reduction of influent BOD concentration. A further comparison with Figure 11 shows no change in effluent BOD concentration during this period.

NON-CANNING SEASON OPERATING CHARACTERISTICS–Figure 13 is a plot of influent BOD and effluent total and soluble BOD vs. time for the dry weather non-canning season. This plot shows a wide fluctuation in influent waste strength, resulting mainly from large variations in flow. However, the effluent BOD concentration, and particularly the soluble portion, maintained a uniform level throughout the period, indicating a stable system.

Influent BOD and effluent total and soluble BOD data are plotted on Figure 14 for the wet weather non-canning season. The influent BOD fluctuated widely as during the dry weather period. Again, this is due principally to the wide variation in flow rate. Effluent total BOD varied to a greater extent than during the dry weather period. However,

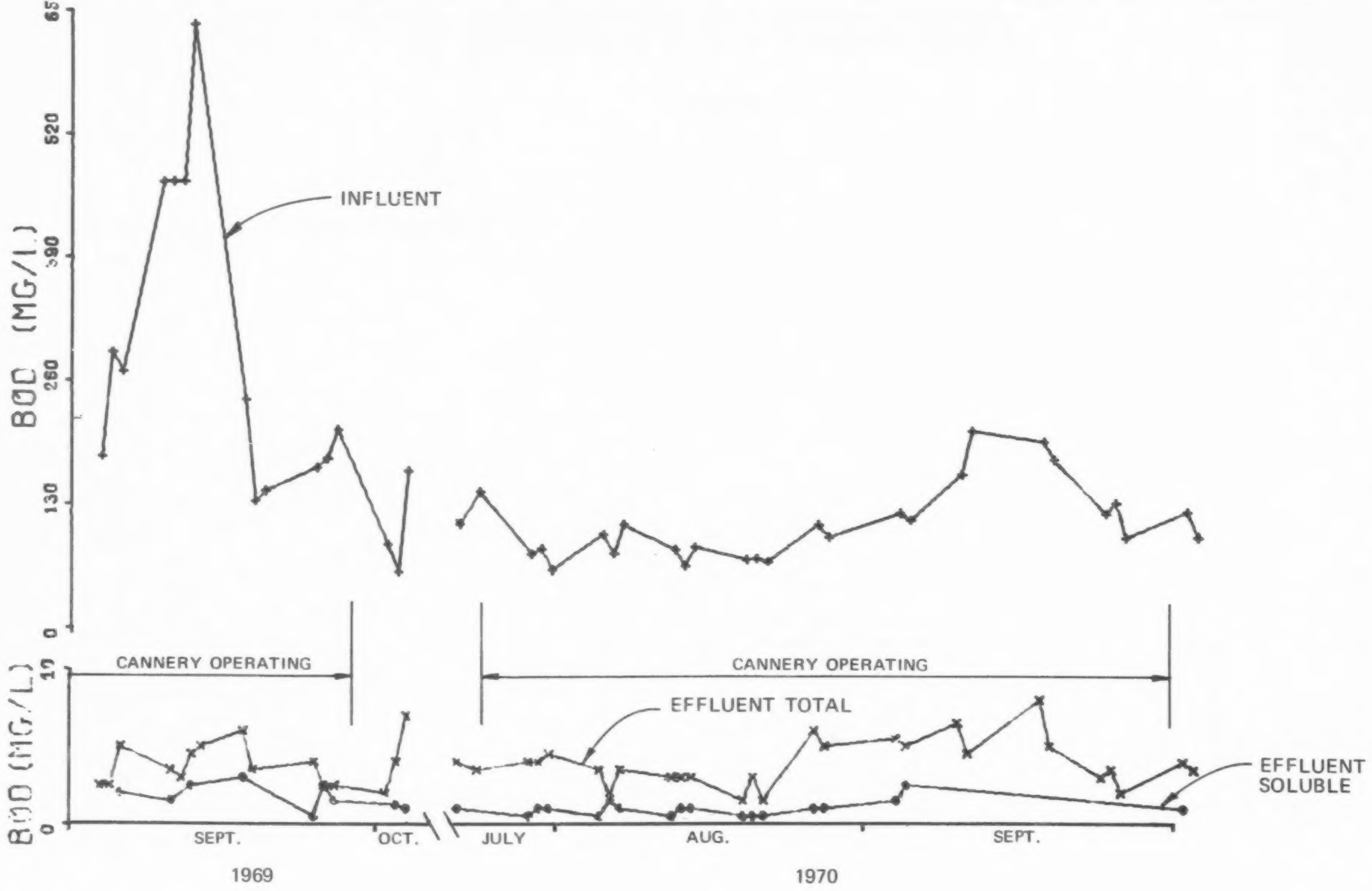


FIGURE 11

INFLUENT BOD AND EFFLUENT TOTAL AND SOLUBLE BOD VS. TIME
CANNING SEASON
(19 JULY 1969 – 27 SEPT 1969, 24 JULY 1970 – 28 SEPT. 1970)

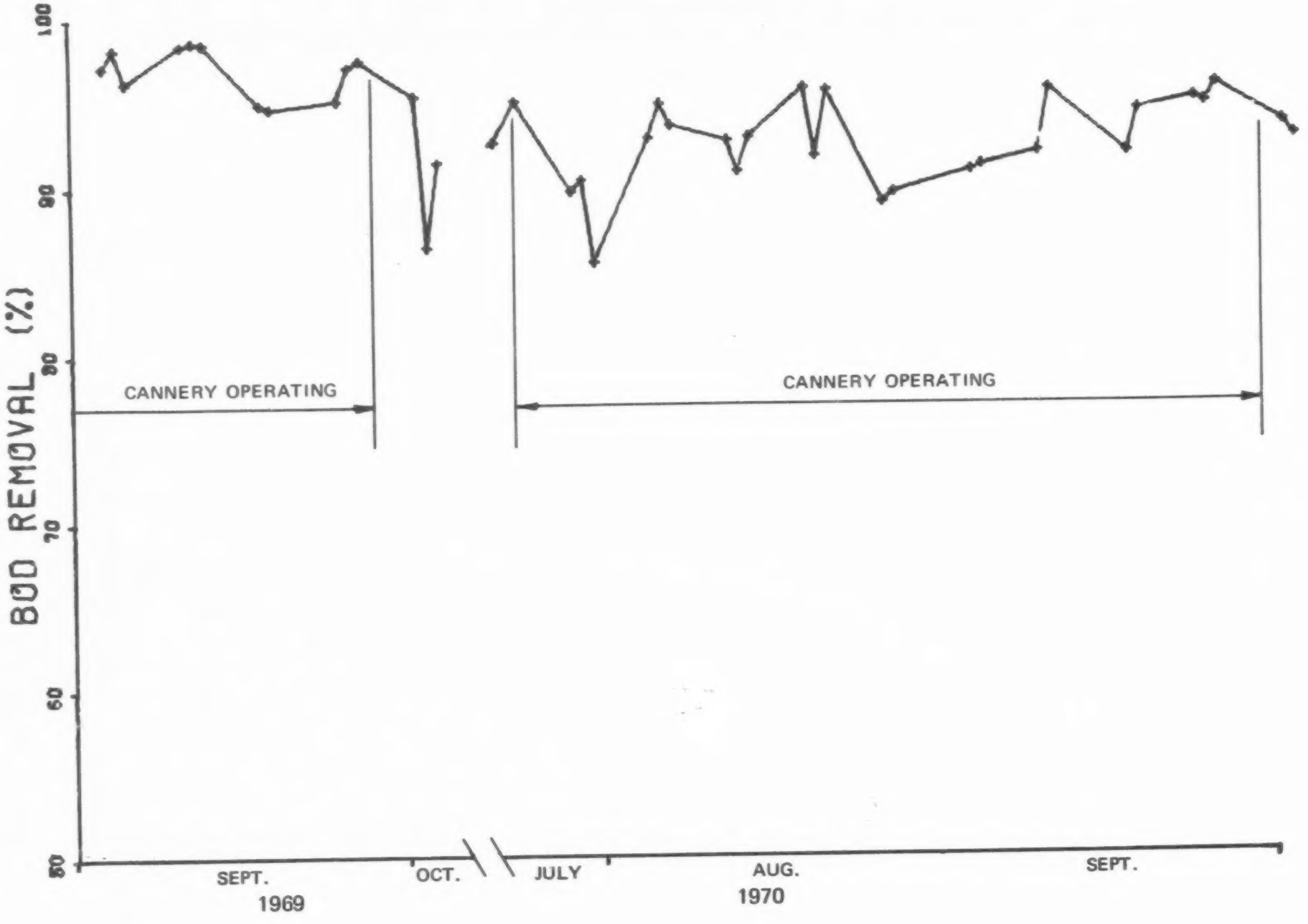


FIGURE 12

BOD REMOVAL VS. TIME – CANNING SEASON
(19 JULY 1969 – 27 SEPT. 1969, 24 JULY 1970 – 28 SEPT. 1970)

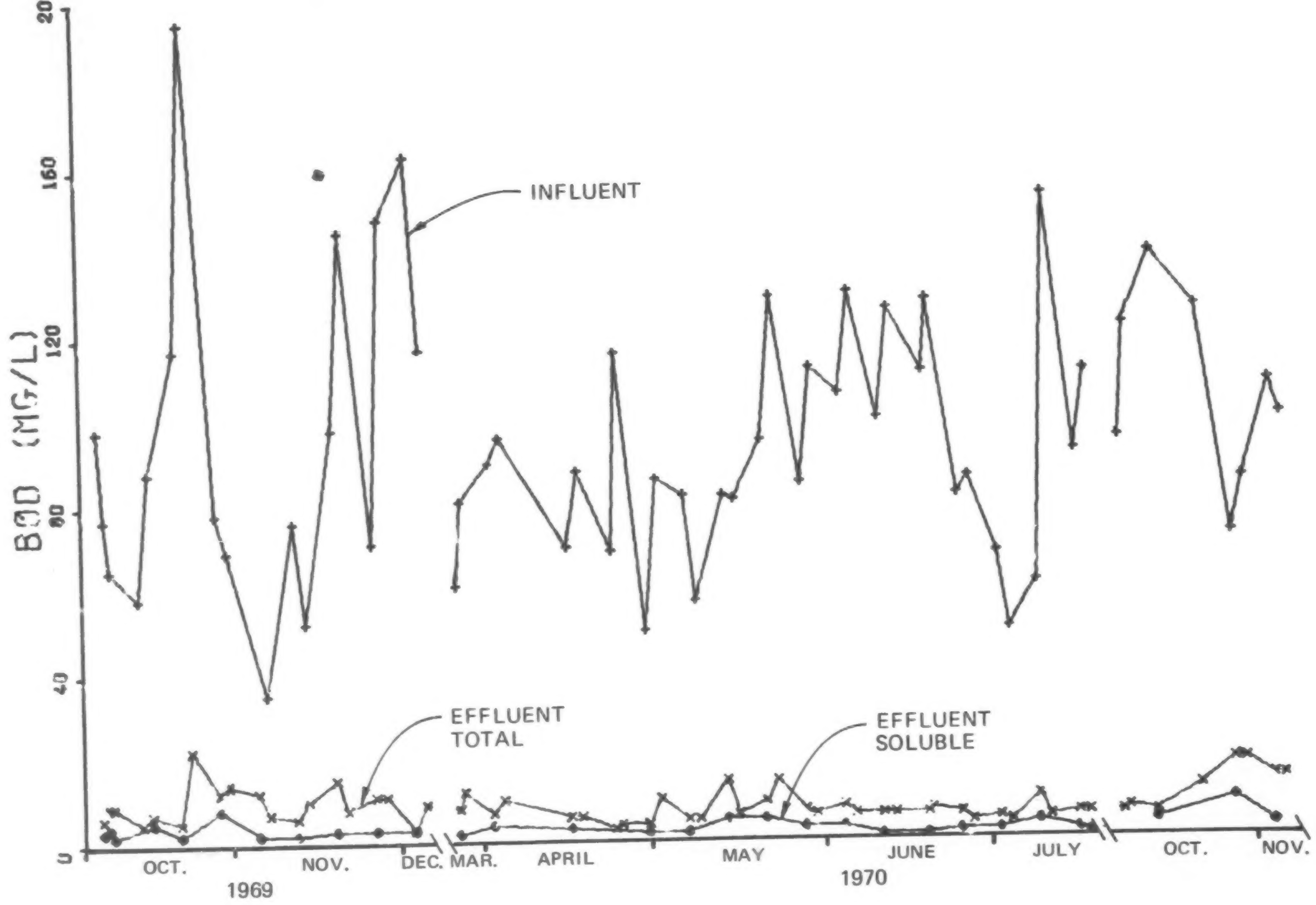


FIGURE 13

INFLUENT BOD AND EFFLUENT TOTAL AND SOLUBLE BOD
VS. TIME – DRY WEATHER NON-CANNING SEASON
(6 OCT. 1960 – 7 DEC. 1969, 23 MARCH, 1970 – 19 JULY 1970, 5 OCT. 1970 – 8 NOV. 1970)

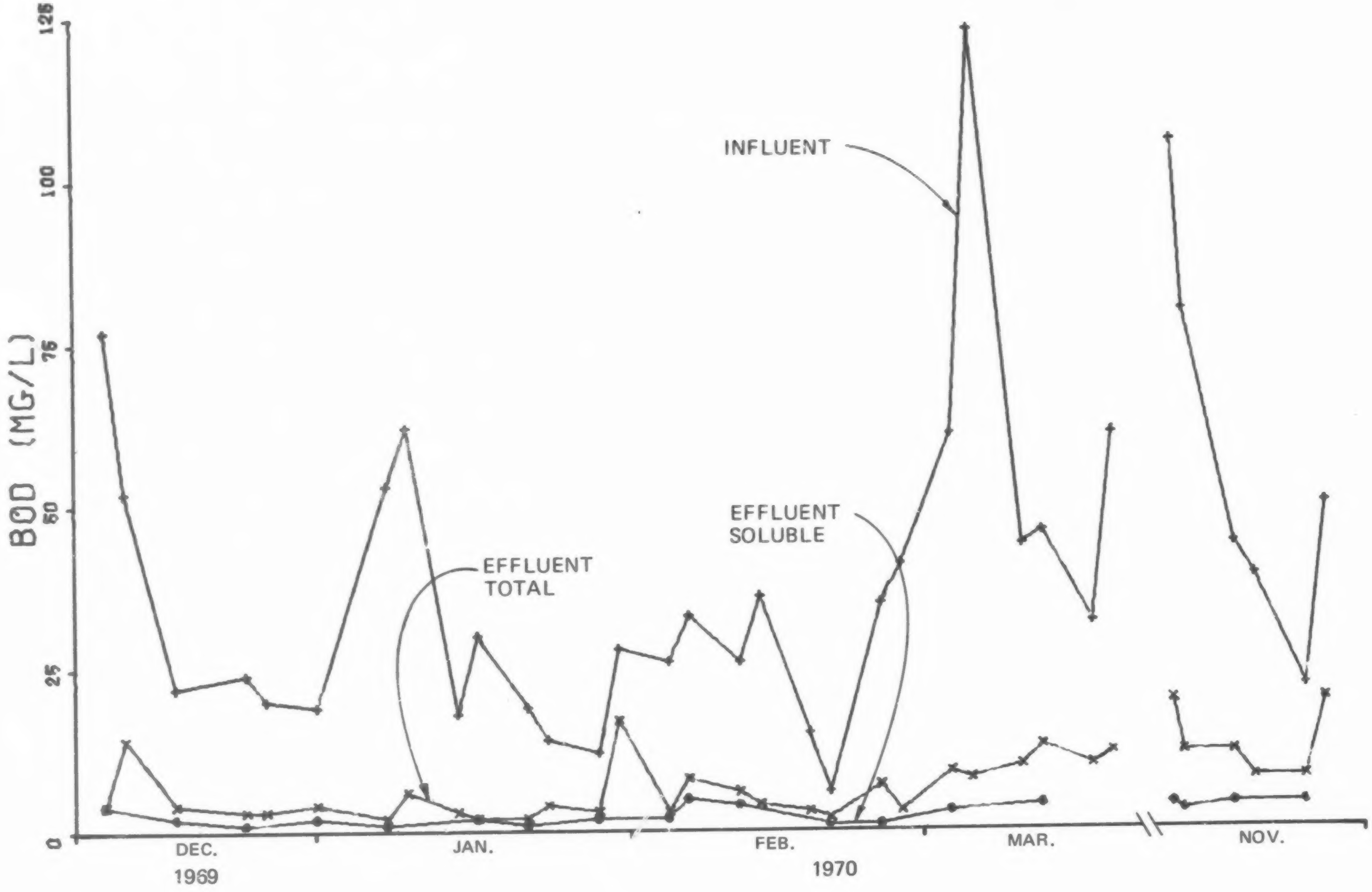


FIGURE 14

INFLUENT BOD AND EFFLUENT TOTAL AND SOLUBLE BOD VS. TIME – WET WEATHER NON-CANNING SEASON
(8 DEC. 1969 – 22 MARCH 1970, 9 NOV. 1970 – 31 NOV. 1970)

effluent soluble BOD was uniform throughout the period, indicating that the variation in effluent quality was mainly due to solids carryover. However, these fluctuations were relatively minor since the effluent total BOD concentration was well below the effluent requirement nearly all the time.

EFFECT OF VARYING DETENTION TIME–Figure 15 is a plot of effluent soluble BOD vs. aeration basin detention time. A significant conclusion is difficult to reach from studying this figure. All data fall within a range of values so small that the difference between the maximum and minimum values is nearly insignificant. The data appear to be quite scattered. However, the scatter occurs over a very small range of soluble BOD values.

From this figure it appears that there is no significant influence on BOD removal efficiency from varying the detention time, at least in the range of loadings encountered in this study.

ORGANIC LOADING–Figure 16 is a plot of organic loading vs. time for the full operating period. In general, the load varied between 0.05 and 0.15 lb BOD per lb MLSS per day, with peak rates as high as 0.4 lb BOD per lb MLSS per day. The SVI averaged 77 for the entire period. The average sludge age for the demonstration period was 19.5 days.

AEROBIC DIGESTION

GENERAL–The aerobic digester was operated continuously throughout the demonstration period. Pumping times to and from the digester were recorded along with the respective solids concentrations. Several other parameters such as pH, D.O., temperature, and alkalinity were monitored to provide a basis for determination of the digester performance.

DIGESTER PERFORMANCE–Solids loading to the digester was much lower than originally anticipated. The average loading rate over the entire period of operation was about 10 lb V.S./day/1000 cu. ft.

Digester performance was evaluated on the basis of the following two parameters: (1) reduction in volatile solids content, and (2) net volatile solids destruction. Both parameters were calculated from average values of the data for the entire demonstration period.

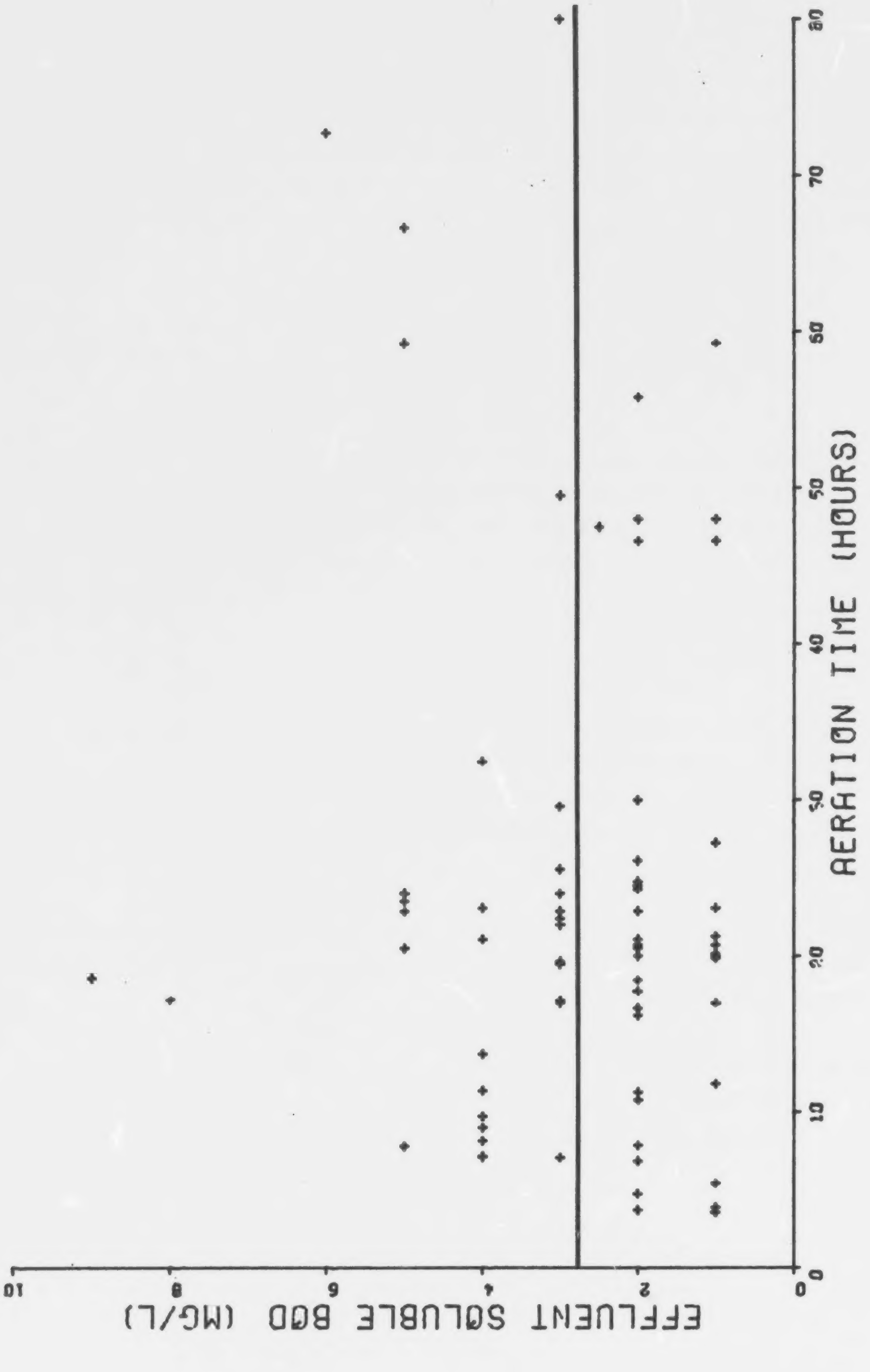


FIGURE 15

EFFLUENT SOLUBLE BOD
VS. AERATION TIME

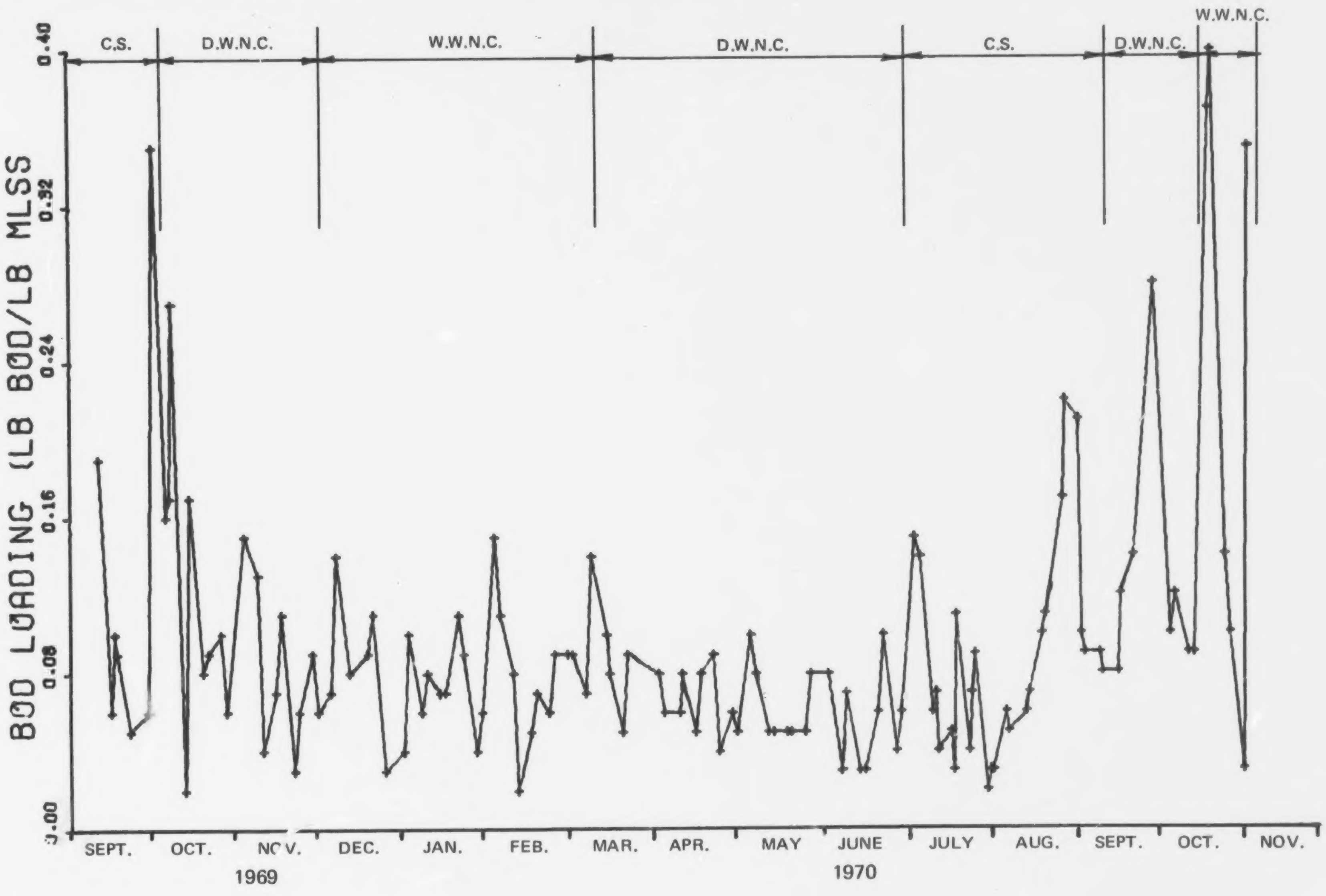


FIGURE 16

ORGANIC LOADING VS. TIME
ALL DATA

Reduction in volatile solids content was calculated by the following formula:

$$\text{VS Reduction (\%)} = 1 - \frac{R_i}{D_i} \times \frac{D_v}{R_v} \times 100$$

where R_i = fraction of inert solids in raw sludge
D_i = fraction of inert solids in digested sludge
D_v = volatile fraction of digested sludge
R_v = volatile fraction of raw sludge

Net reduction of volatile solids was based on the ratio of total pounds of volatile solids removed from the digester to total pounds of volatile solids applied corrected for change in volatile solids concentration in the digester and quantity of volatile solids applied to fill the digester initially.

The average reduction (in percentage of total) of volatile solids content for the demonstration period was 5 percent. The net volatile solids destruction for the same period (calculated in pounds) was about 34 percent. The average digester detention time was 69 days. The basin temperature averaged about 12 degrees C and the pH averaged 5.1 with an average alkalinity of 17 mg/l.

QUANTITY AND CHARACTER OF DIGESTED SLUDGE–The quantity of sludge wasted to the humus ponds was calculated from the data recorded for pumping time and solids concentration of the digested sludge. The total quantity of digested sludge wasted to the humus ponds during the demonstration period was about 265,000 pounds (dry solids basis), or an average of 670 lb/day. The average volatile solids content of this sludge was about 63 percent.

The relative drying character of the digested sludge was observed during the early summer of 1970, when the supernatant was drained from the sludge in one of the humus ponds. Some odor was present for a short period of time when the sludge was exposed to the air. However, the odor, much like that of a barnyard although less intense, was not objectionable and disappeared with the formation of a crust on the sludge layer.

The wet sludge, which had a maximum depth of about 1.5 feet, required a period of about 3 months to dry completely. The maximum depth of sludge in the pond after drying was about 6 inches. The sludge formed individual clumps upon drying, as shown by a photograph of the dried sludge included in Appendix A. The relatively long period required for complete drying of the sludge was primarily due to inability to drain out the water which was trapped underneath the sludge blanket.

The length of time before removal of the solids from the humus storage pond will be required was estimated to be a minimum of three years, based on the quantity of solids accumulated in one humus storage pond during the study period.

SOLIDS DEPOSITS

Aeration basin number 1 was drained after about 14 months of operation, to determine the quantity of solids which had accumulated in the basin. Prior to dewatering the basin, the biological solids were washed out by recycling water from the polishing pond back through the plant, without returning activated sludge to the aeration basin. Pumping was begun after reducing the MLSS level as low as possible. All four aerators were run while pumping, to keep the residual MLSS in suspension.

A visual observation of the condition of the basin bottom was made. Figure 17 is a sketch of aeration basin number 1, showing the condition of the bottom, after 14 months operation. Appendix A contains photographs of the basin, showing bottom scour and solids deposits.

Bottom scour was evident around the concrete target pads beneath the aerators. The maximum depth of scour was about 15 inches.

Solids deposition was generally light. The greatest accumulation occurred adjacent to the aerator location in the northeast corner of the basin. The maximum depth of solids in this location was about 1.5 feet. The solids consisted mainly of non-degradable plastic materials and appeared to contain a relatively small amount of grit. The overall quantity of solids deposited was relatively insignificant. Removal of solids from the basin should not be required for a period of at least five years, based on these observations.

VELOCITY PROFILES

Velocity measurements were made in Aeration Basin No. 1 under different operating conditions. Typical examples of velocity profiles are shown in Figures 18, 19 and 20, (plans at different depths) and Figure 21 (section) for one set of operating conditions. Velocity profiles for other operating conditions are included in Appendix C. The numbers shown on the aerators indicate the operating horsepower of each unit.

Measurements were made from a boat using a Gurley No. 622 propeller-type current meter. These data provide an evaluation of the mixing characteristics in the basin at different power levels and provides an indication of possible low velocity areas where solids settling might occur.

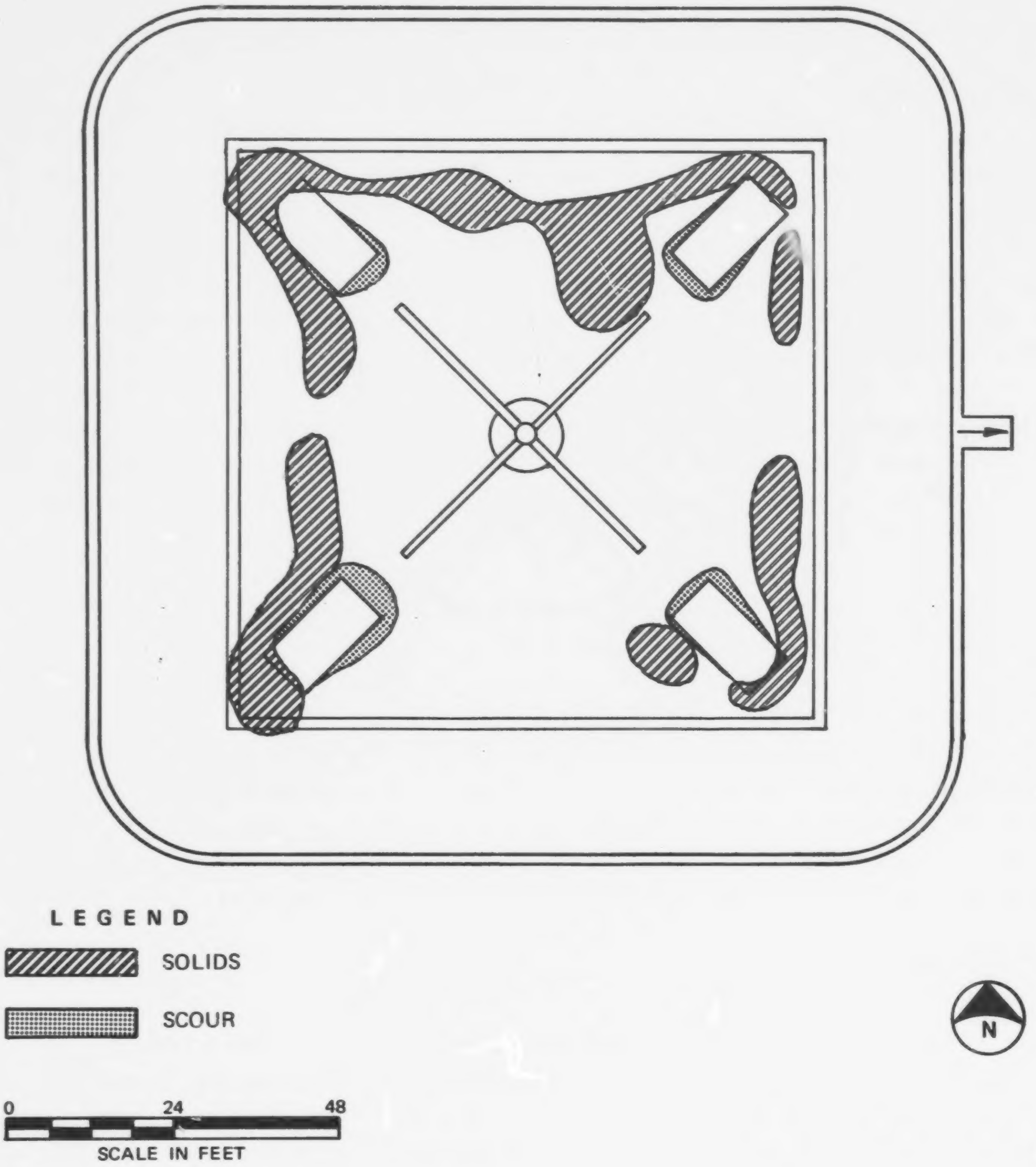
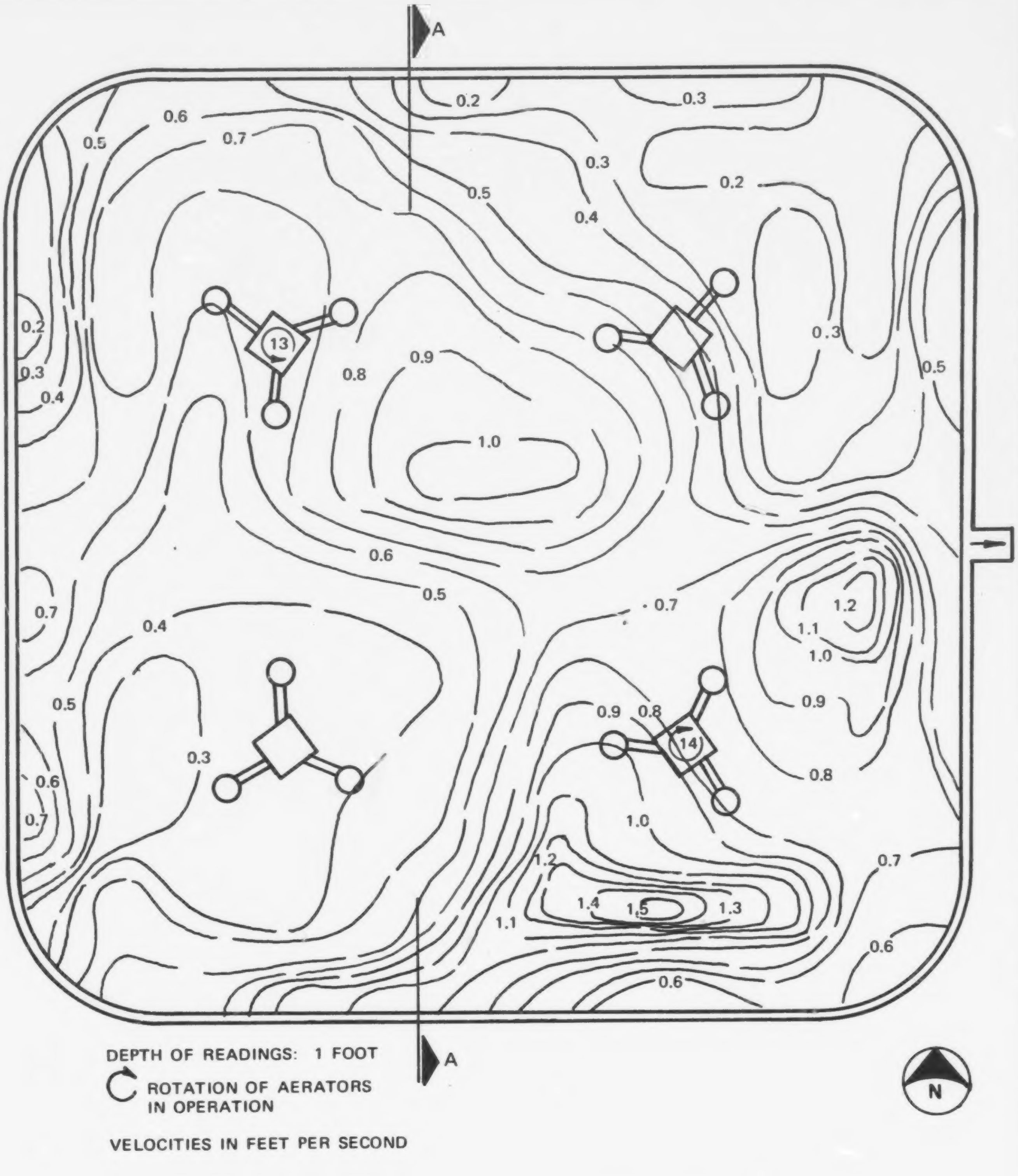

LEGEND
SOLIDS
SCOUR
N
0
24
48
SCALE IN FEET

FIGURE 17

SOLIDS ACCUMULATION IN AERATION
BASIN NO. 1 – OCTOBER 1970


A
0.2
0.3
0.6
0.5
0.7
0.3
0.5
0.2
0.4
0.2
13
0.3
0.4
0.9
0.8
0.3
0.5
1.0
0.6
0.5
0.7
1.2
0.7
1.1
0.4
1.0
0.5
0.9
0.8
0.9
0.3
14
0.6
0.8
0.7
1.0
1.2
0.7
1.4
1.5
1.3
1.1
0.6
0.6
A
N

DEPTH OF READINGS: 1 FOOT

ROTATION OF AERATORS IN OPERATION

VELOCITIES IN FEET PER SECOND

FIGURE 18

AERATION BASIN VELOCITY PROFILE
TWO AERATORS – LOW SPEED

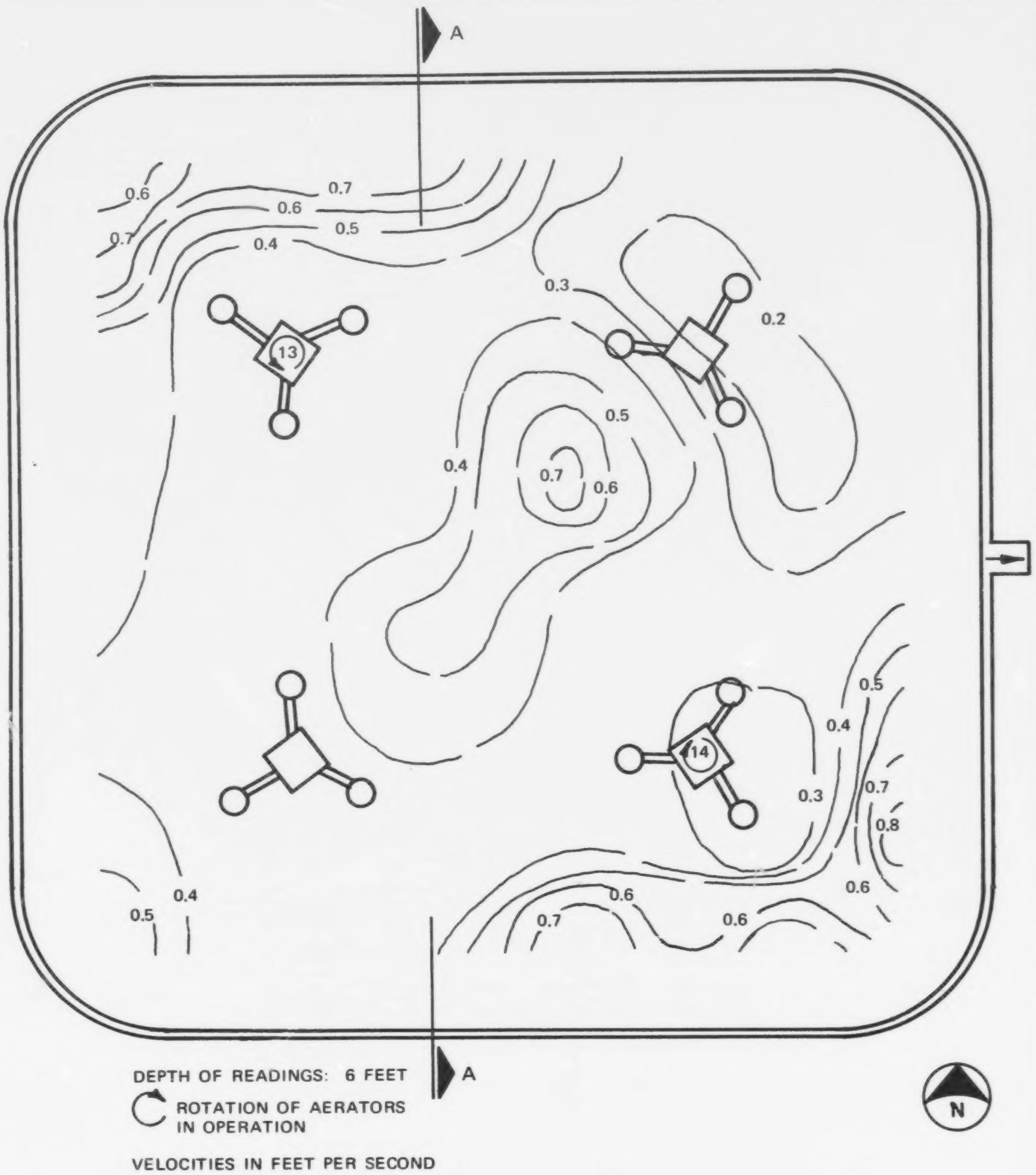

A
0.6
0.7
0.6
0.5
0.7
0.4
0.3
0.2
13
0.5
0.4
0.7
0.6
0.5
0.4
14
0.3
0.7
0.8
0.6
0.4
0.5
0.6
0.7
0.6
A
N
DEPTH OF READINGS: 6 FEET
ROTATION OF AERATORS IN OPERATION
VELOCITIES IN FEET PER SECOND

FIGURE 19

AERATION BASIN VELOCITY PROFILE
TWO AERATORS – LOW SPEED

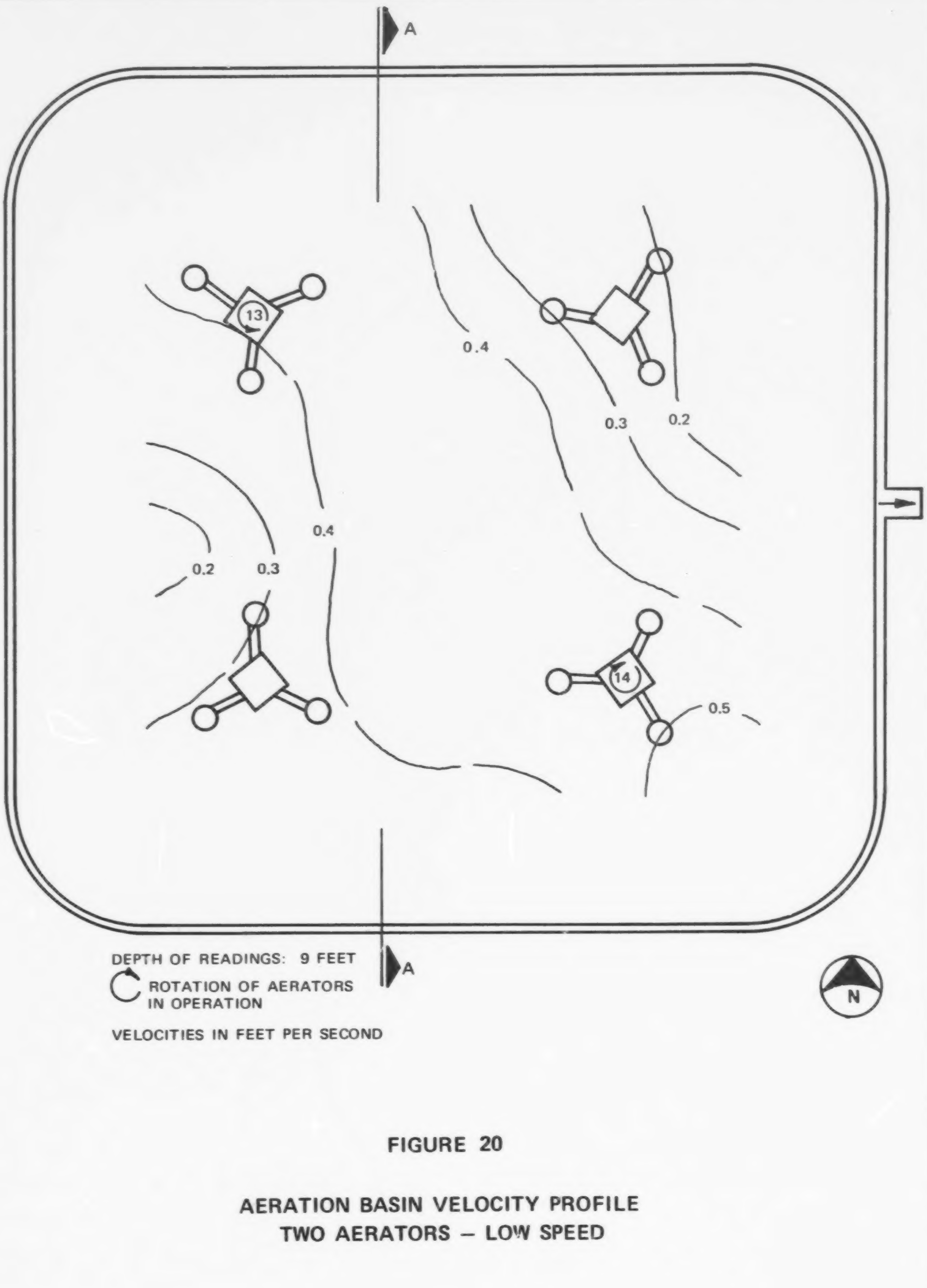
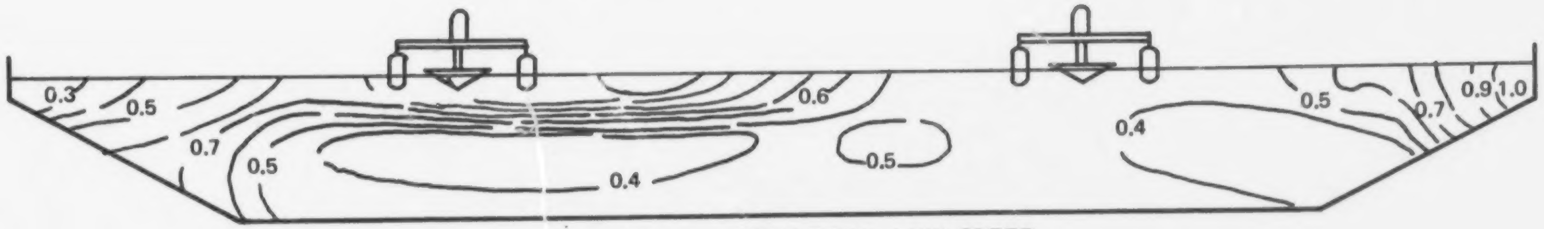
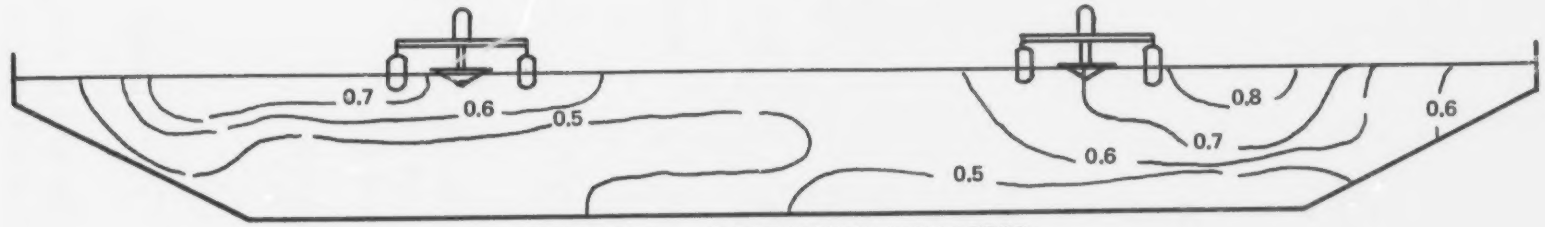

A
13
0.4
0.3
0.2
0.4
0.2
0.3
14
0.5
A
DEPTH OF READINGS: 9 FEET
ROTATION OF AERATORS IN OPERATION
VELOCITIES IN FEET PER SECOND
N

FIGURE 20

AERATION BASIN VELOCITY PROFILE
TWO AERATORS – LOW SPEED


0.3
0.5
0.7
0.5
0.4
0.6
0.5
0.4
0.5
0.7
0.9
1.0

SECTION A – 2 AERATORS, LOW SPEED


0.7
0.6
0.5
0.5
0.6
0.7
0.8
0.6

SECTION B – 4 AERATORS, LOW SPEED

FIGURE 21

AERATION BASIN
VELOCITY PROFILES

Analysis of the profiles for the minimum operating conditions (two aerators - low speed) shows bottom velocities in the range of 0.4 to 0.5 fps, found by Eckenfelder [7] to be a minimum for uniform suspension of solids.

DISSOLVED OXYGEN PROFILES

Dissolved oxygen (D.O.) measurements were taken at different depths in aeration basin number 1 under different operating conditions. The D.O. readings were taken from a boat using a YSI meter and field probe. Figures 22, 23 and 24 show D.O. profiles at different depths in the basin and Figure 25 is a D.O. profile section through the basin.

These figures show a higher D.O. concentration toward the south side of the basin. Ammeter readings indicated a greater current draw and thus a higher horsepower output from the aerators on that side of the basin. This difference is indicated by the operating horsepower shown on each aerator. This resulted in a greater transfer of oxygen to the mixed liquor and, therefore, higher D.O. readings.

OXYGEN UPTAKE RATES

Several oxygen uptake measurements were taken on grab samples of aeration basin and aerobic digester mixed liquors. The tests were conducted by the following procedure: A two-liter bottle was filled with the sample, a YSI D.O. probe was inserted and the top of the bottle was sealed. D.O. readings were taken at regular intervals until the oxygen was depleted. The sample was mixed continuously during the test by a magnetic stirrer with teflon stirring bar.

The oxygen uptake data was plotted as dissolved oxygen used vs. time. These figures are included in Appendix C.

The aeration basin oxygen uptake rates varied from 0.155 to 1.03 mg O_2 per day per mg MLVSS at a temperature range of 17.5 to 19 degrees C. The aerobic digester oxygen uptake rate averaged about 0.012 mg O_2 per day per mg MLVSS. No detailed analysis of the system oxygen requirements was made because not enough data were available to establish a relationship between oxygen uptake and BOD removals.

PVC LINER

The original demonstration plan involved the use of PVC (polyvinyl chloride) sheet liner in the aeration basins and aerobic digester. The City subsequently elected to use shotcrete liners in these units instead of PVC, at an additional cost of about $0.25 per square foot. However, since one of the grant objectives was to evaluate the use of PVC as a lining

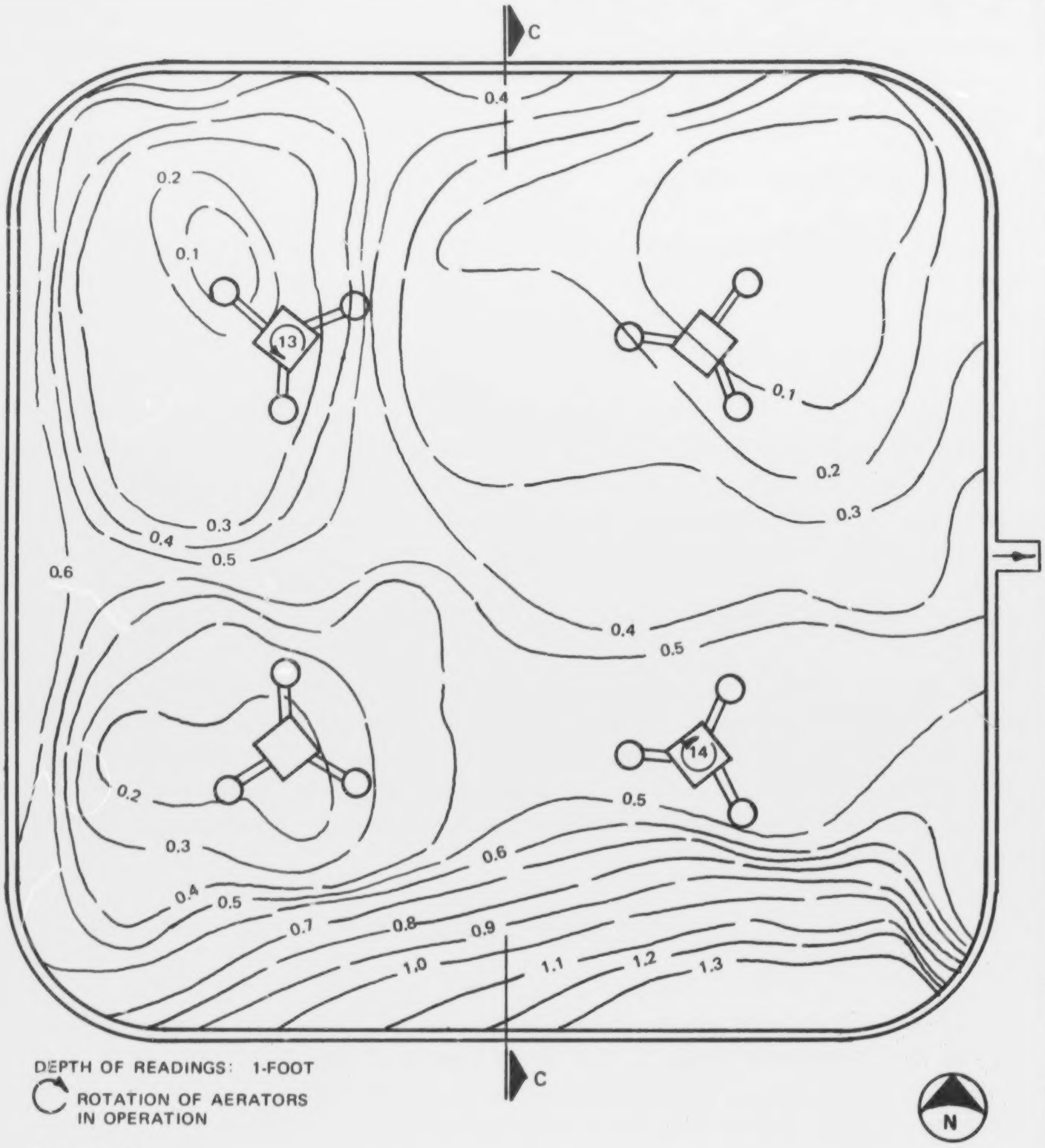
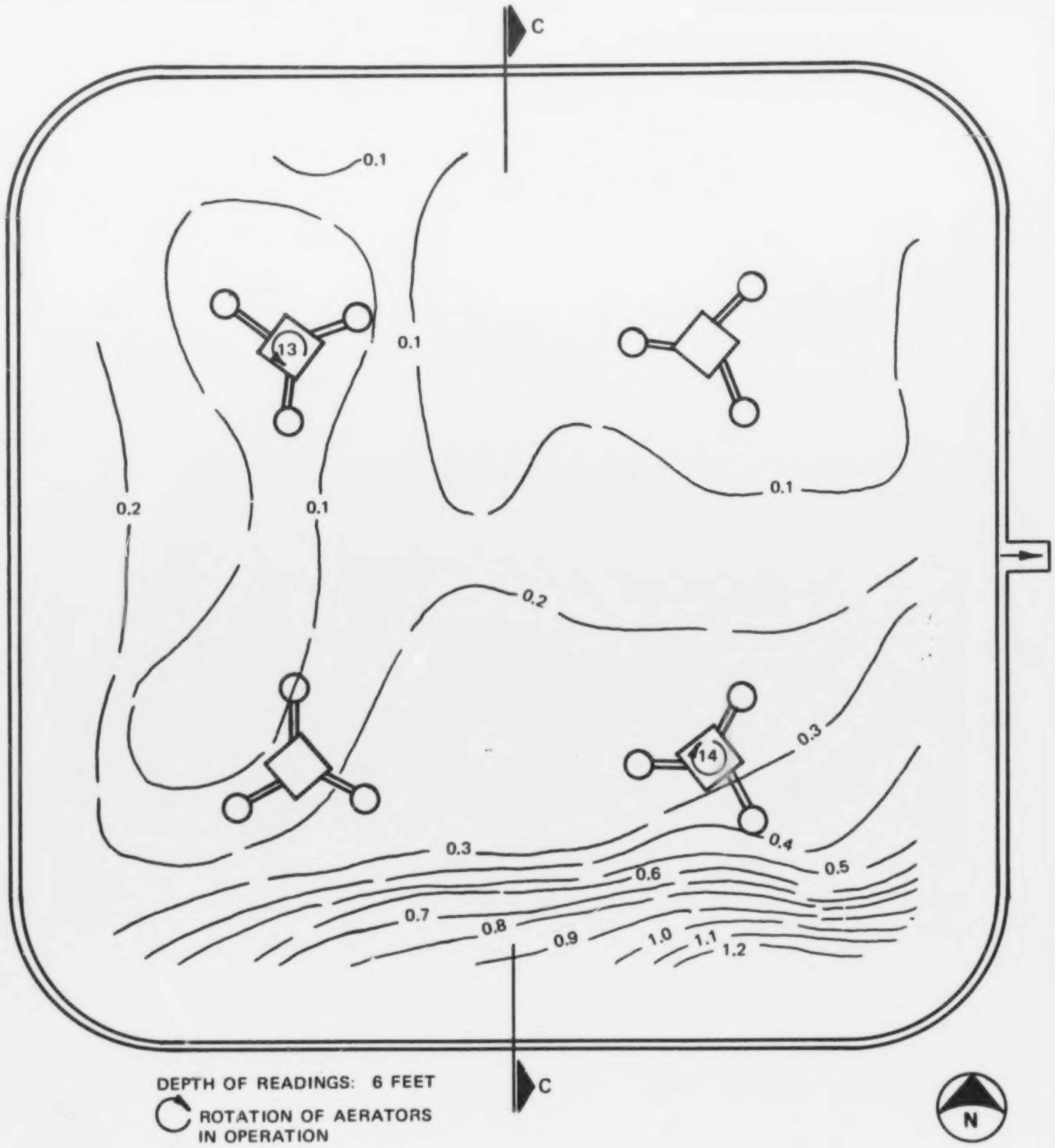

C
0.4
0.2
0.1
13
0.1
0.2
0.3
0.3
0.4
0.5
0.6
0.4
0.5
14
0.2
0.5
0.3
0.6
0.4
0.5
0.7
0.8
0.9
1.0
1.1
1.2
1.3
C
DEPTH OF READINGS: 1-FOOT
ROTATION OF AERATORS IN OPERATION
D.O. IN MG/L
N

FIGURE 22

AERATION BASIN D.O. PROFILE
TWO AERATORS – LOW SPEED


C
0.1
0.1
13
0.2
0.1
0.1
0.2
0.3
14
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
1.2
C
N

DEPTH OF READINGS: 6 FEET

ROTATION OF AERATORS IN OPERATION

D.O. IN MG/L

FIGURE 23

AERATION BASIN D.O. PROFILE
TWO AERATORS – LOW SPEED

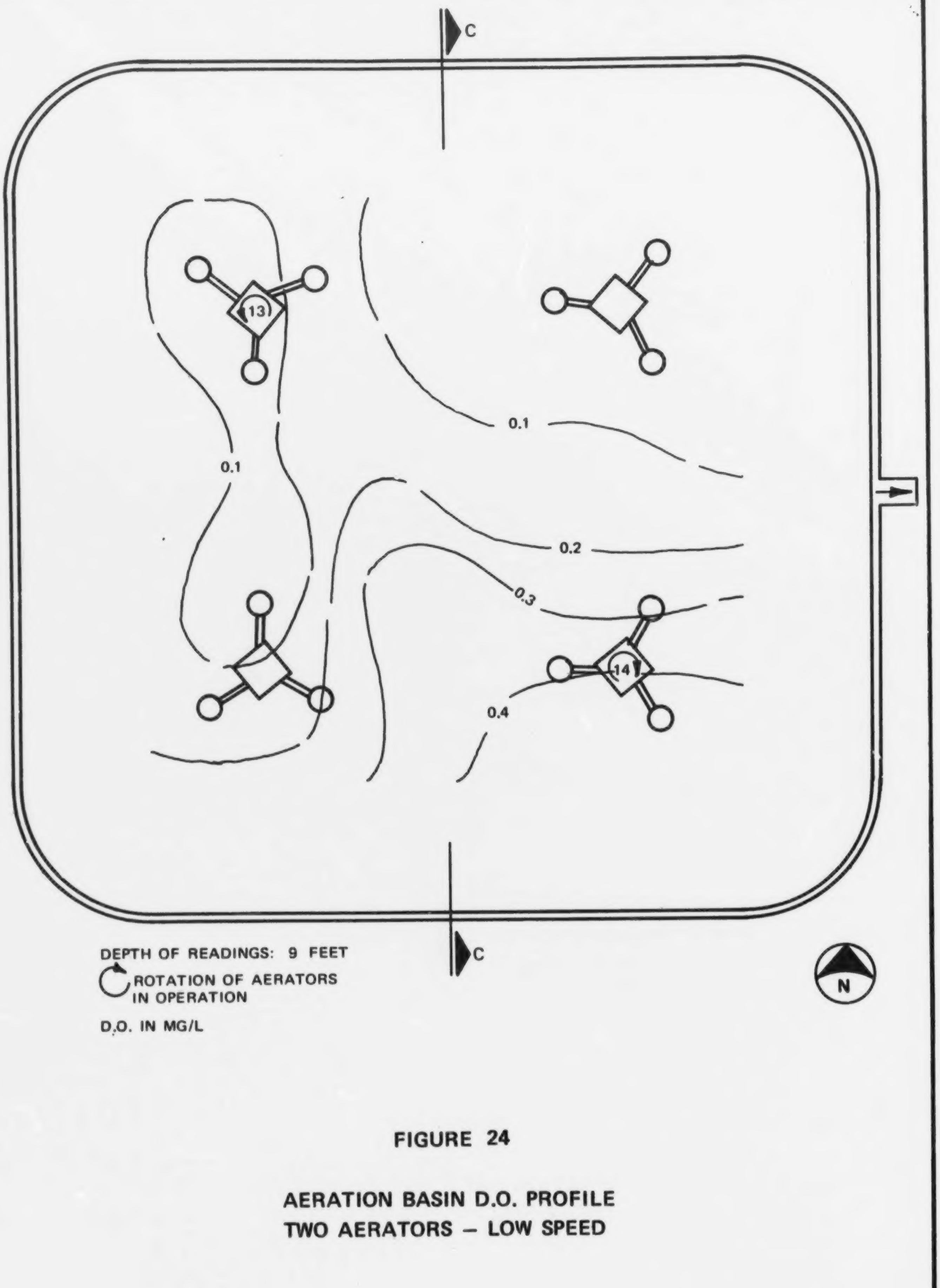
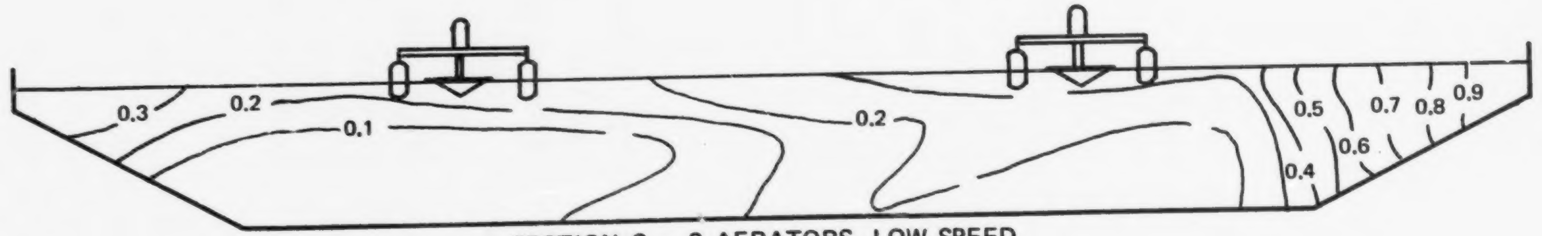
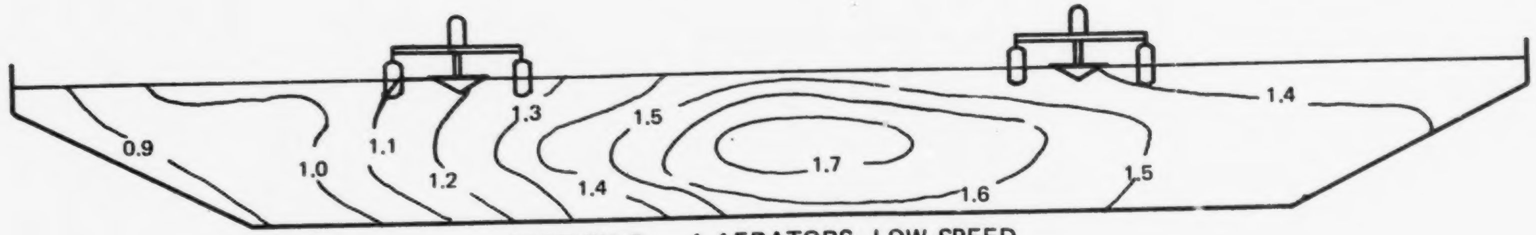

C
13
0.1
0.1
0.2
0.3
14
0.4
C
DEPTH OF READINGS: 9 FEET
ROTATION OF AERATORS IN OPERATION
D.O. IN MG/L
N

FIGURE 24

AERATION BASIN D.O. PROFILE
TWO AERATORS – LOW SPEED


0.3
0.2
0.1
0.2
0.5
0.7
0.8
0.9
0.6
0.4

SECTION C – 2 AERATORS, LOW SPEED


1.4
0.9
1.0
1.1
1.2
1.3
1.5
1.4
1.7
1.6
1.5

SECTION D – 4 AERATORS, LOW SPEED

FIGURE 25

AERATION BASIN
D.O. PROFILES

material, several PVC strips were placed in the aeration basin, to be removed and tested at periodic intervals. The first specimens were removed after one year of submergence, and the following tests were performed: Grabes Tear Test (ASTM D1000-4) and tensile and elongation tests (ASTM D882). The tests were performed by the General Engineering Department of Oregon State University.

The tests showed no detrimental effect on the physical properties of the PVC material. The complete test results are included in Appendix E.

OPERATIONAL CONSIDERATIONS

Relatively few operating problems were encountered. Those affecting plant operation were mechanical problems involving equipment. These are described in the following paragraphs.

AERATION EQUIPMENT–Shortly after plant startup, a bearing failure occurred in one of the mechanical aerator gearmotors. This necessitated startup of the other aeration basin. Subsequent operation resulted in similar failures in four of the eight remaining aerators. The cause of this problem was isolated as a failure to lubricate these bearings during assembly of the units at the factory.

RECYCLED SLUDGE PUMP STATION–A delay in delivery of these pumps resulted in use of a temporary pumping facility. Very little control of the sludge recirculation rate was possible for a period of several weeks.

WASTE SLUDGE PUMPS–A problem with the waste sludge pump motors was encountered which limited control of the sludge wasting rate. The problem, which involved motor overheating and shutdown, was corrected by replacing the motors on both pumps. The sludge wasting rate was affected for an 8-month period.

Another problem developed which affected sludge wasting rate. A leak in the pump suction line allowed air to enter the pump, thus reducing the pump capacity. This condition affected operation for a period of about 2 months. The sludge pump automatic time clock failed to operate properly on several occasions, resulting in a rapid reduction in MLSS concentration in the aeration basin.

AUTOMATIC SAMPLERS–A mechanical problem with the automatic sampler timing device required manual sampling of the influent and effluent flows during the first 4 months of operation. The sample quality suffered somewhat as a result of this problem even though the samples were manually mixed to form composites during the operating shift.

SECTION VIII
FINANCIAL CONSIDERATIONS

CONSTRUCTION COSTS

The total capital cost for construction of the treatment facility, including engineering and land costs, was $605,470. A detailed cost breakdown is included in Appendix D. Comparison of this total with a total cost of $913,000 for a conventional activated sludge plant, obtained from Smith's curves [13] and updated to 1968, indicates a substantial saving in capital costs.

OPERATION AND MAINTENANCE COSTS

The total operation and maintenance costs for the first year of operation were $32,700. Operation and maintenance costs for the plant, after deducting costs attributable only to the demonstration program, were $22,200 and represent the normal yearly cost for running the plant. The added costs due to the demonstration program were attributed to additional sampling and testing requirements, supervision and data analysis. A detailed breakdown of operation and maintenance costs is included in Appendix D.

Michel [14] reported annual operation and maintenance costs, for activated sludge plants treating an average daily flow of 1.0 mgd, ranging from about $21,000 to $70,000 with an average cost of about $37,000 per year (costs updated to 1970). The operation and maintenance cost of $22,200 for the Dallas plant was about 60 percent of the average.

Smith [13] reported average operation and maintenance costs, for activated sludge plants treating an average daily flow of 1.0 mgd, of about $100 per million gallons treated (costs updated to 1970). Operation and maintenance costs at the Dallas plant averaged $33 per million gallons treated or about 33 percent of Smith's figure.

TOTAL ANNUAL COSTS

Table 7 lists total annual costs of the treatment system for the first year of operation. Total annual costs include: 1) annual capital amortization costs (6 percent interest, 20 years), 2) annual operation and maintenance costs, and 3) equipment replacement and depreciation costs (5 percent of equipment capital cost).

TABLE 8
TOTAL ANNUAL COSTS

Item	Annual Cost
Capital Cost	$605,470
Amortized Capital Cost (6%-20 years)	52,800
Annual Operation and Maintenance Cost	22,200
Estimated Equipment Replacement and Depreciation	6,800
TOTAL ANNUAL COST	$ 81,800

On the basis of this total annual cost, and the total pounds of BOD removed during the year, cost of treatment was calculated as $0.258 per pound of BOD removed.

This unit cost appears to be higher than would normally be expected. However, the plant has been operated far below the BOD loading capacity and high infiltration into the collection system results in a very dilute waste during wet weather periods. Therefore, the cost per unit of BOD is increased because of the required operation of units not related to BOD removal.

A more realistic approach would be to calculate unit cost for the amortized capital cost portion of the annual cost, based on available BOD loading capacity rather than on BOD removal. The unit cost, calculated on this basis, was $0.097 per pound of BOD removed, and included $0.026 per pound of BOD based on debt service and depreciation and $0.071 per pound of BOD removed based on operation and maintenance costs.

SECTION IX
DISCUSSION

ACTIVATED SLUDGE SYSTEM

MICROBIOLOGY–The system was operated at a relatively long sludge age, which tends to favor the higher forms of organisms. Periodic microscopic examinations revealed the presence of a wide range of organisms, ranging from bacteria through rotifers, during different periods throughout the demonstration program.

The system appeared to operate most efficiently when a moderately large population of rotifers and a large population of stalked ciliates were present in the activated sludge. The moderate rotifer population indicated that the system was very stable, as discussed in Section III.

Very few filamentous organisms were observed, although some nitrification occurred in the aeration basins throughout the period. However, the alkalinity of the wastewater was sufficient to buffer the system, thus minimizing the conditions favorable to filamentous growth.

The predominant protozoa identified in the activated sludge during the periods of most efficient operation were the stalked ciliates *Vorticella campanula* and *Vorticella microstoma.*

SUBSTRATE REMOVAL–Substrate utilization in the activated sludge system, as previously discussed in Section III, is described by the following equation:

$$kS_e = \frac{S_r}{X_a t}$$

The substrate removal rate, k, is equal to the slope of the first order curve passing through the origin, on a plot of total pounds of BOD removed per day per pound of MLVSS vs. mg/l of effluent soluble BOD. Figure 26 is a plot of substrate removal vs. effluent soluble BOD for all data obtained during the demonstration period.

The field data plotted on Figure 26 show considerable scatter, as may be expected. For this reason, the curve from the origin was drawn through the mean coordinates of the data points.

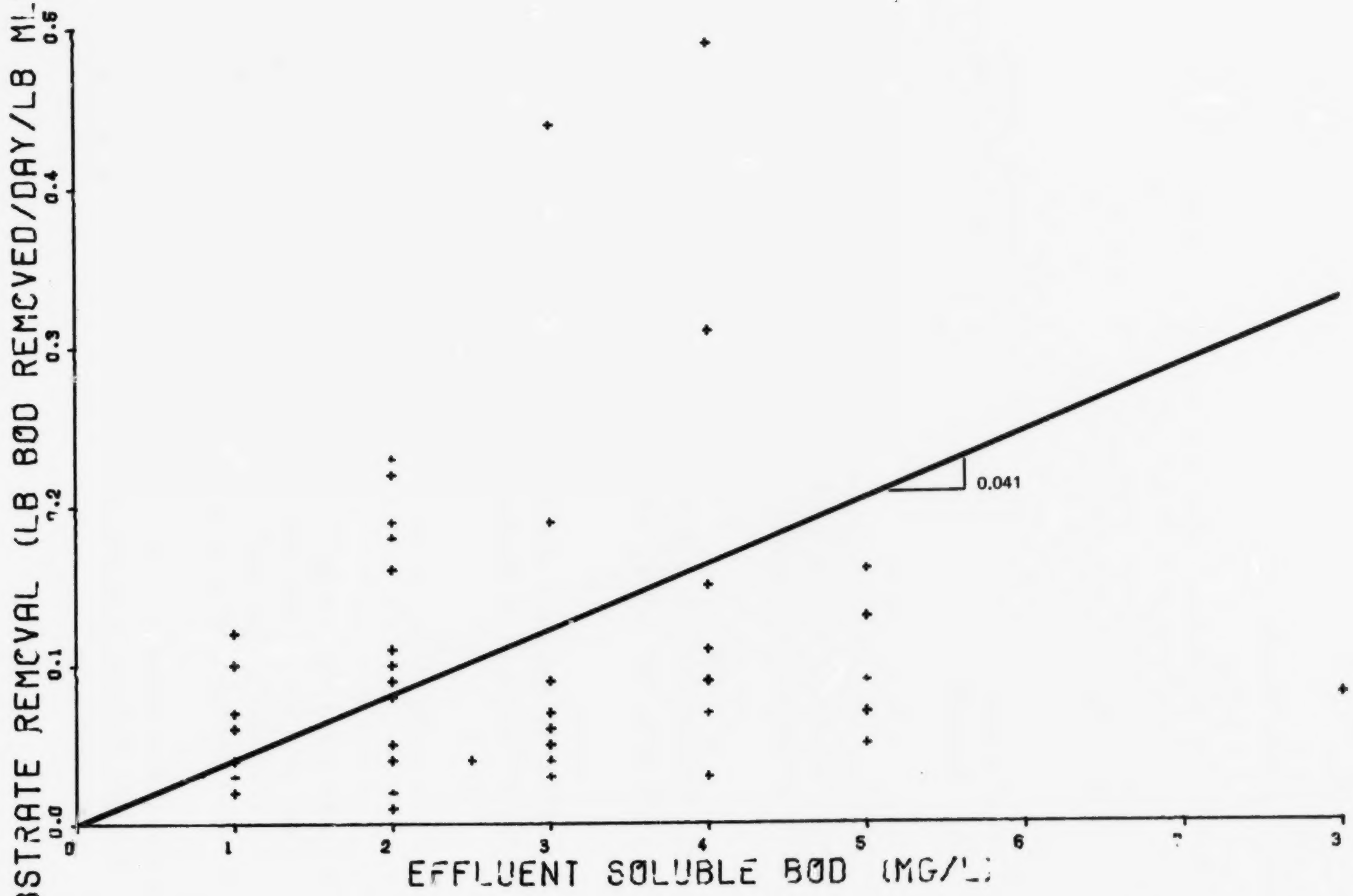


FIGURE 26

SUBSTRATE REMOVAL VS. EFFLUENT SOLUBLE BOD

This figure includes data for a relatively wide range of temperatures (9 degrees C - 24 degrees C). Three separate curves were plotted initially, for data of three different temperature ranges. Substrate removal rate coefficients obtained from these plots, however, showed no reasonable variation with temperature as described in the literature [15, 16, 17]. Therefore, it was concluded that, within the range of operation experienced, the substrate removal coefficient is not affected significantly by temperature.

The average substrate removal coefficient, k, calculated from the data plotted on Figure 26, was 0.041. This value agrees closely with the data for domestic wastewater presented by Eckenfelder [15], but is somewhat higher than the coefficients obtained for potato processing wastewater [17] and fruit processing wastewater [16]. This indicates that the composition of the wastewater influent to the treatment plant at Dallas is predominantly domestic in nature.

CANNERY STARTUP, OPERATION AND SHUTDOWN—The discussion of the nature of the influent wastewater presented in the previous paragraph is reflected by the lack of effect on plant operation of cannery startup and shutdown.

The peak BOD loads to the system occur during the canning season. However, no significant change in effluent quality has resulted from these peaks. The gradual increase and decrease of processing load at the cannery tends to reduce the shock load effect of cannery startup and shutdown.

The distance from the cannery to the treatment plant may have some effect on the strength of the canning waste. It is likely that a significant BOD reduction occurs in the sewer between the cannery and the treatment plant since the canning waste is highly soluble and the detention time is relatively long.

The canning waste is also substantially diluted by the domestic wastewater and by infiltration into the system, resulting in a lower BOD concentration at the treatment plant.

EFFLUENT QUALITY—The effluent quality data presented in Section VII give an indication of the flexibility and stability of the system. The effluent quality was consistently good during a wide range of operating conditions. The wide variation in flow through the plant resulted in a large fluctuation of aeration time. During the wet weather flows, aeration times were as low as 3.4 hours.

The organic loading rate, in general, varied between 0.05 and 0.15 lb BOD per lb MLSS per day. This range of loading rates is somewhat lower than the values generally considered desirable for efficient operation of activated sludge systems. However, a check of the effluent BOD values plotted on Figure 9 shows that an effluent of extremely good quality was maintained at these low loadings. The organic loading was particularly low during the wet weather non-canning season due to the low influent BOD. Some control of the organic loading was accomplished by reducing the MLSS level. However, a minimum MLSS level of 700 mg/l was maintained to provide a resistance to shock loads from glue waste discharges and to minimize washout of solids during peak flow periods.

Aeration basin temperatures varied from 9 degrees C to 24 degrees C. The low temperature periods generally coincided with periods of low organic loadings and short aeration times, resulting in conditions highly unfavorable for biological treatment. However, the stability of the system prevented the occurrence of a poor-quality effluent.

CLARIFIER OPERATION–Figure 27 is a plot of clarifier hydraulic loading vs. effluent suspended solids. Clarifier overflow rates varied from about 200 gallons per day per square foot to as high as 2500 gallons per day per square foot during wet weather periods. The scatter of the data on this plot shows that there is very little correlation between clarifier efficiency and hydraulic loading rate. However, efficiency of the clarifier was good at all ranges of loading, indicating a reasonably stable activated sludge system.

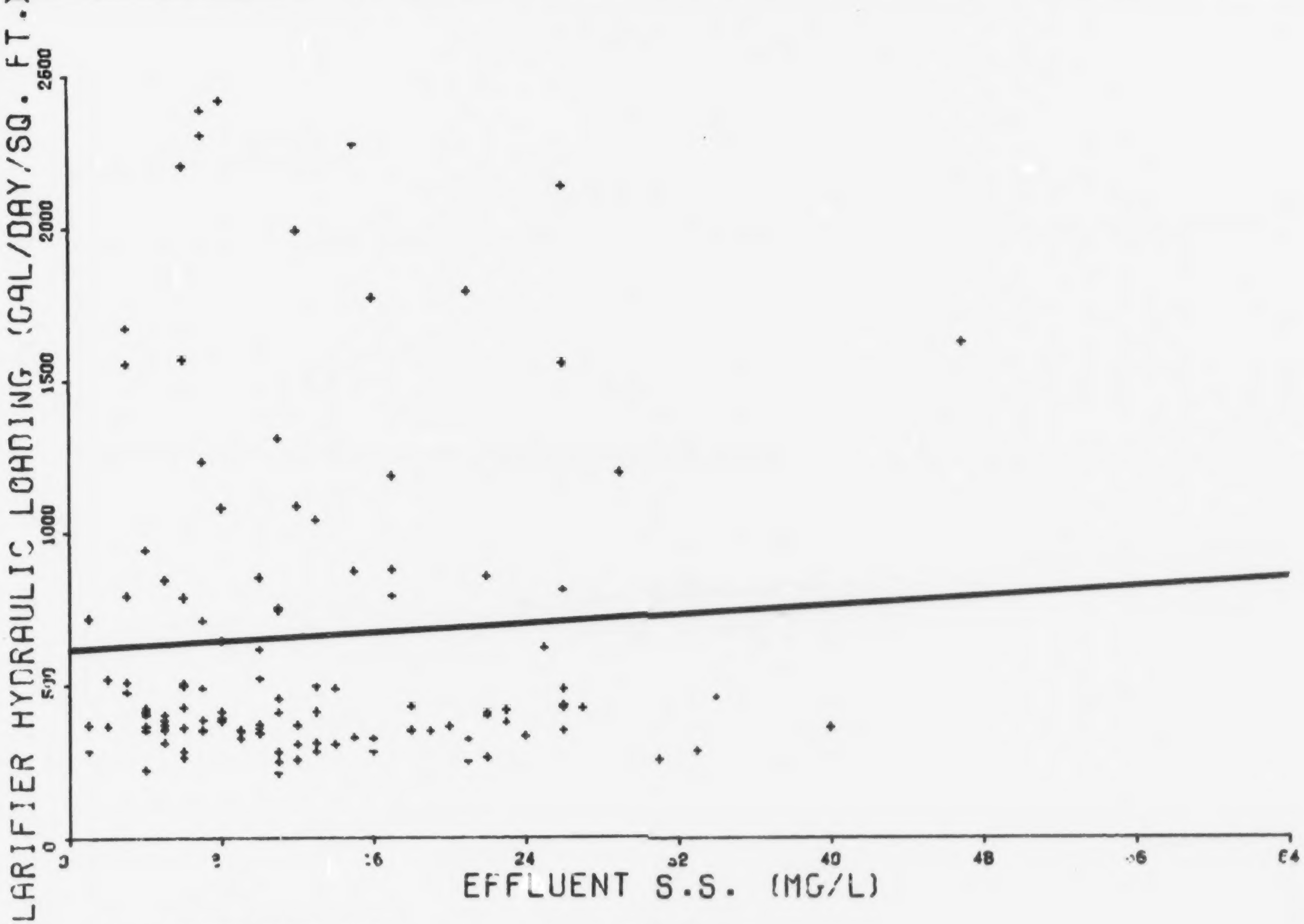
Clarifier solids loadings were plotted vs. effluent suspended solids on Figure 28. These loadings ranged from a low of about 4 pounds per day per square foot to as much as 50 pounds per day per square foot. Analysis of this figure yields the same conclusions as discussed in the preceding paragraph for hydraulic loading.

OXYGEN TRANSFER–Each aerator is capable of transferring to water, at standard conditions, a minimum of 1180 pounds of oxygen per day at high speed and at least 50 percent of that amount on low speed.

Figure 29 is a plot of operating horsepower vs. oxygen transfer capacity. Listed on this curve are the points of operation possible to achieve with the aeration equipment available. This indicates the wide range of flexibility of the system.

AEROBIC DIGESTION

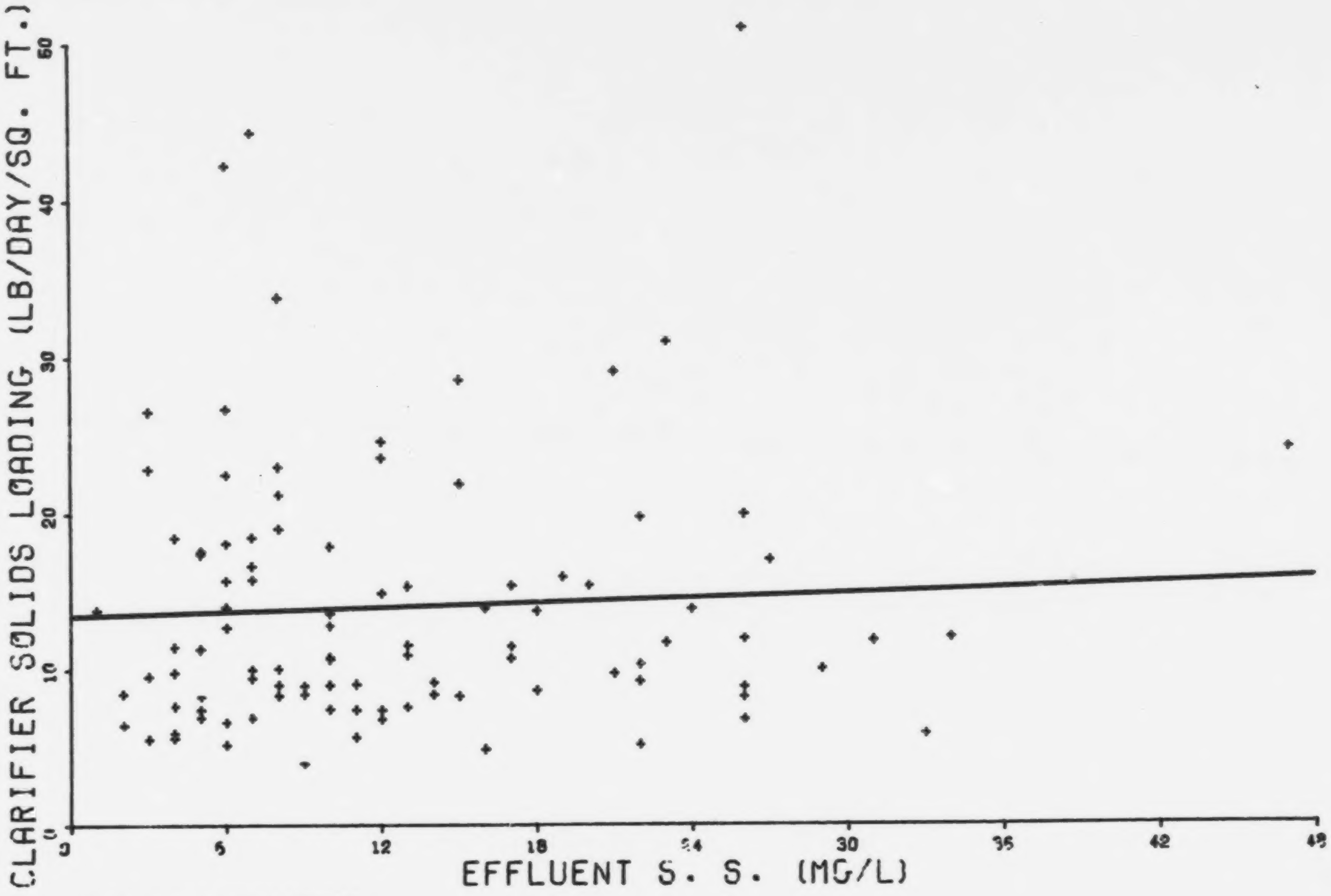
VOLATILE SOLIDS REDUCTION–The average reduction (in percentage) of volatile solids content of the waste sludge was about 5 percent. The long sludge age (19 days average) in the activated sludge system contributed to this low percentage of volatile reduction as indicated by Loehr [18]. Another factor affecting the reduction in volatile content was the relatively low average VSS content of the waste sludge (63 percent).



NOTE: CURVE FIT BY METHOD OF
LEAST SQUARES

FIGURE 27

CLARIFIER HYDRAULIC LOADING
VS. EFFLUENT SUSPENDED SOLIDS



NOTE: CURVE FIT BY METHOD
OF LEAST SQUARES

FIGURE 28

CLARIFIER SOLIDS LOADING VS.
EFFLUENT SUSPENDED SOLIDS

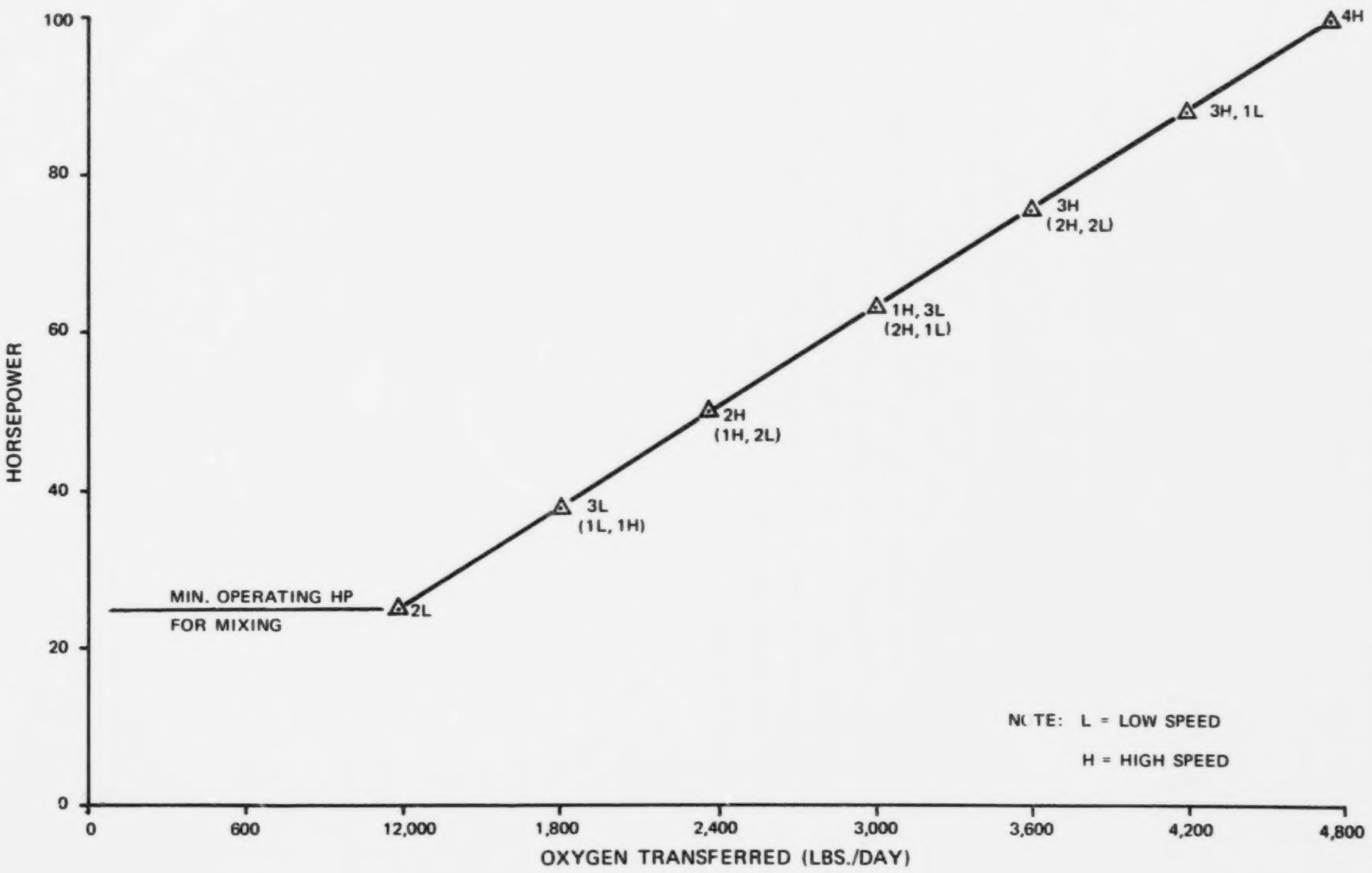


FIGURE 29

HORSEPOWER – OXYGEN TRANSFER RELATIONSHIPS

An analysis of the sludge wasted to the digester was made, using information obtained from the literature and concepts discussed by Loehr [18]. Figure 30 shows the calculated composition of the waste sludge solids. The solids lost from the system per day consisted of an average of 950 pounds pumped to the digester and 220 pounds in the effluent, for a total of 1170 pounds per day.

A large portion of the influent TSS is carried through the system as residual, nondegradable, solids. These residual solids consisted of all the inert solids plus that portion of the volatile solids which were nondegradable.

The nondegradable influent volatile solids were assumed to constitute 40 percent of the influent VSS, as suggested by Eckenfelder [15].

The remainder of the solids wasted to the digester (387 lb/day) were biological solids. The biological solids were also broken down into three classifications: inert, nondegradable volatile, and degradable volatile. The fraction of each type depends on two important operating parameters: sludge age and temperature. Analysis of data presented by Eckenfelder [20] and McKinney [3] indicates that, at 15.7 degrees C and a sludge age of 19.5 days, biological solids will be about 77 percent volatile. Determination of the degradable and nondegradable portions of the biological volatile solids was based on data presented by McCarty and Brodersen [10]. The biological volatile solids were estimated to be about 60 percent degradable at a temperature of 15.7 degrees C and assuming a 19.5 day sludge age.

This analysis shows that there is a maximum percentage of the total waste solids that are degradable for a particular set of operating conditions. For the average operating conditions encountered during this study, the maximum degradable portion of the waste solids was calculated to be about 19 percent.

This low percentage of degradable solids is partially due to the relatively high proportion of influent suspended solids to waste activated sludge solids in the sludge wasted to the digester. The long sludge age of the system is also partially responsible for this condition, since the biological solids are already highly degraded before being wasted to the digester.

The items discussed in the preceding paragraphs serve to explain the low percentage of reduction in volatile content through the digester. The reduction in volatile content should increase in the future, as digester biological solids loading increases and the sludge age decreases.

DETENTION TIME–The minimum digester detention time occurring during the demonstration period was about 35 days. A plot of VSS content of the digested sludge vs. digester detention time showed highly scattered data with no reasonable trends.

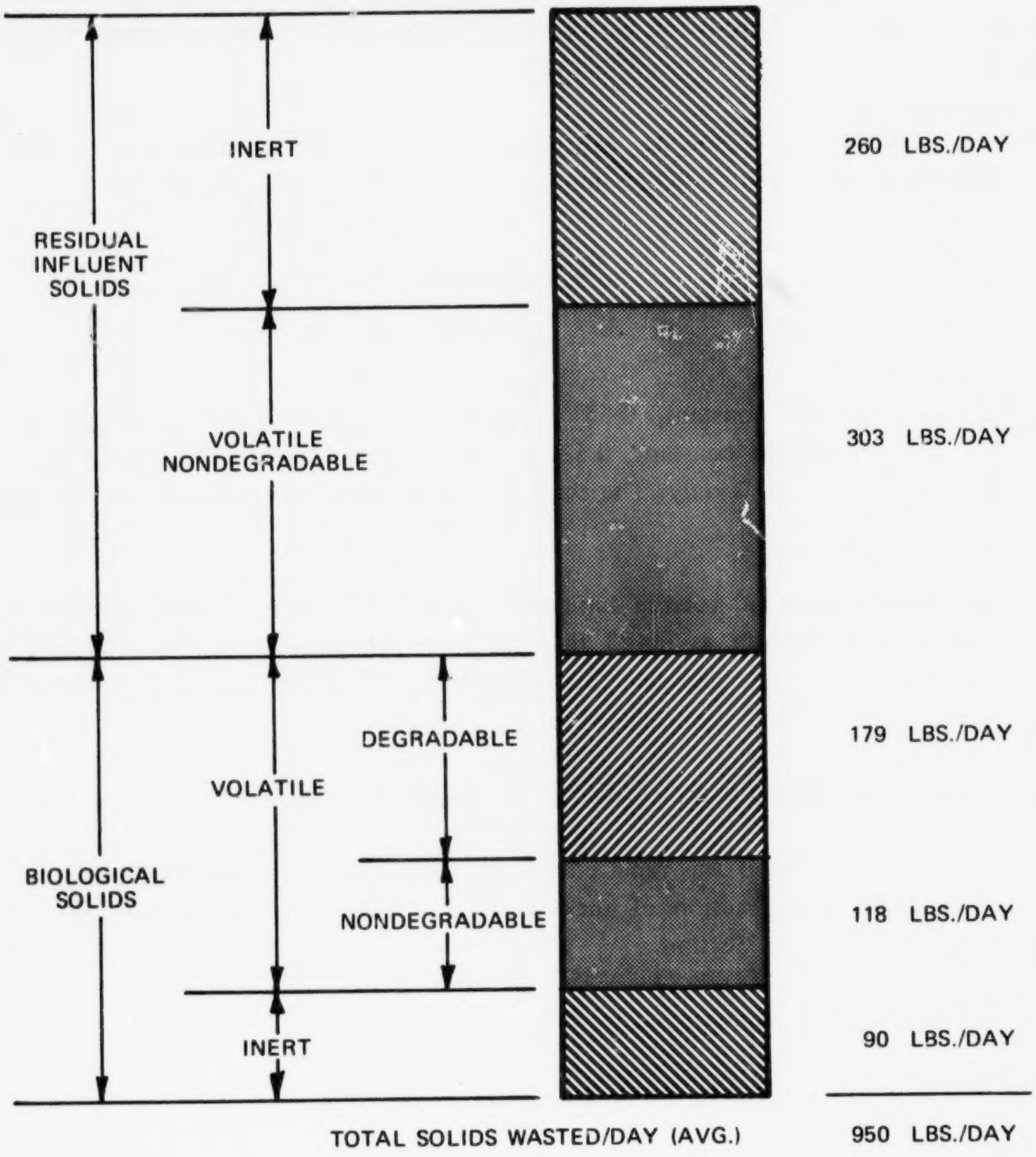


FIGURE 30

COMPOSITION OF WASTE SLUDGE SOLIDS

Lawton and Norman [19] have shown that very little volatile solids reduction occurs at detention times greater than about 15 days. Since the minimum detention time recorded during this study period was 35 days, all the data lay in the range of low activity. Thus, the data scatter is a result of variations in testing rather than differences in VSS reduction.

SLUDGE YIELD–The following equation, giving the net accumulation of volatile biological solids per pound of BOD removed, was presented in Section III:

$$\frac{A}{F} = 0.65 - \frac{0.53}{1 + \frac{E}{bM}}$$

In this equation, the constant, 0.65, represents the sludge yield coefficient, a, for domestic sewage and the constant, 0.53, is the yield coefficient based on synthesis of degradable biological solids only. The term E/M is equal to the reciprocal of the sludge age, T_s.

The net accumulation of volatile biological solids per pound of BOD removed was calculated from field data as 0.425 lb VSS per lb BOD removed. The equation was modified as follows, to enable the determination of the sludge yield coefficient, a, for this system:

The constant 0.65 was replaced by the constant a.

The constant 0.53 was replaced by the constant 0.6a, which represents the degradable fraction of the volatile bioligical solids as established previously in this Section.

The term E/M was replaced by $1/T_s$.

These modifications resulted in the following expression for a:

$$a = \frac{A/F}{1 - \frac{0.6}{1 + \frac{1}{bT_s}}}$$

This equation yields an "a" value of 0.70 for a sludge age of 19.5 days and a "b" equal to 0.1, which is in relatively close agreement with McCarty and Brodersen's a value of 0.65 for domestic sewage.

CHLORINE CONTACT

The effluent total coliform count averaged about 41 per 100 ml with a maximum of 230 and a minimum of 2. A chlorine residual of about 1.0 ppm was maintained in the effluent nearly all the time.

Studies were conducted, using Rhodamine B dye and a fluorometer, to determine the actual detention time in the chlorine contact channel. These tests indicated that the actual detention time was about 83 percent of the theoretical detention time, based on the time of peak dye concentration. The efficiency could likely be increased substantially by the addition of an inlet baffling system. However, the coliform data has been well within the required standards, indicating that the channel has provided effective contact time.

SECTION X
ACKNOWLEDGMENTS

This project was supported in part by Environmental Protection Agency Research and Development Grant No. 11060 EZR. Appreciation is expressed to the administration and staff of the City of Dallas, Oregon, the EPA Pacific Northwest Water Laboratory personnel, and Cornell, Howland, Hayes & Merryfield staff members for their cooperation and assistance during this study.

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SECTION XII
PUBLICATIONS

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Graham, J.L. and Filbert, J.W., "An Unconventional Approach," *The American City*, October 1970.

SECTION XIII
ABBREVIATIONS

mg/l	Milligrams per liter
BOD	Biochemical Oxygen demand
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
mgd	Million gallons per day
TSS	Total suspended solids
VSS	Volatile suspended solids
SVI	Sludge volume index
D.O.	Dissolved oxygen
O_2	Oxygen
T_s	Sludge age

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SECTION XIV
APPENDIXES

NO.		PAGE
A	Photographs	87
B	Process and Laboratory Equipment	95
C	Velocity and Dissolved Oxygen Data	99
D	Costs	115
E	Results of PVC Tests	119

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APPENDIX A

PHOTOGRAPHS

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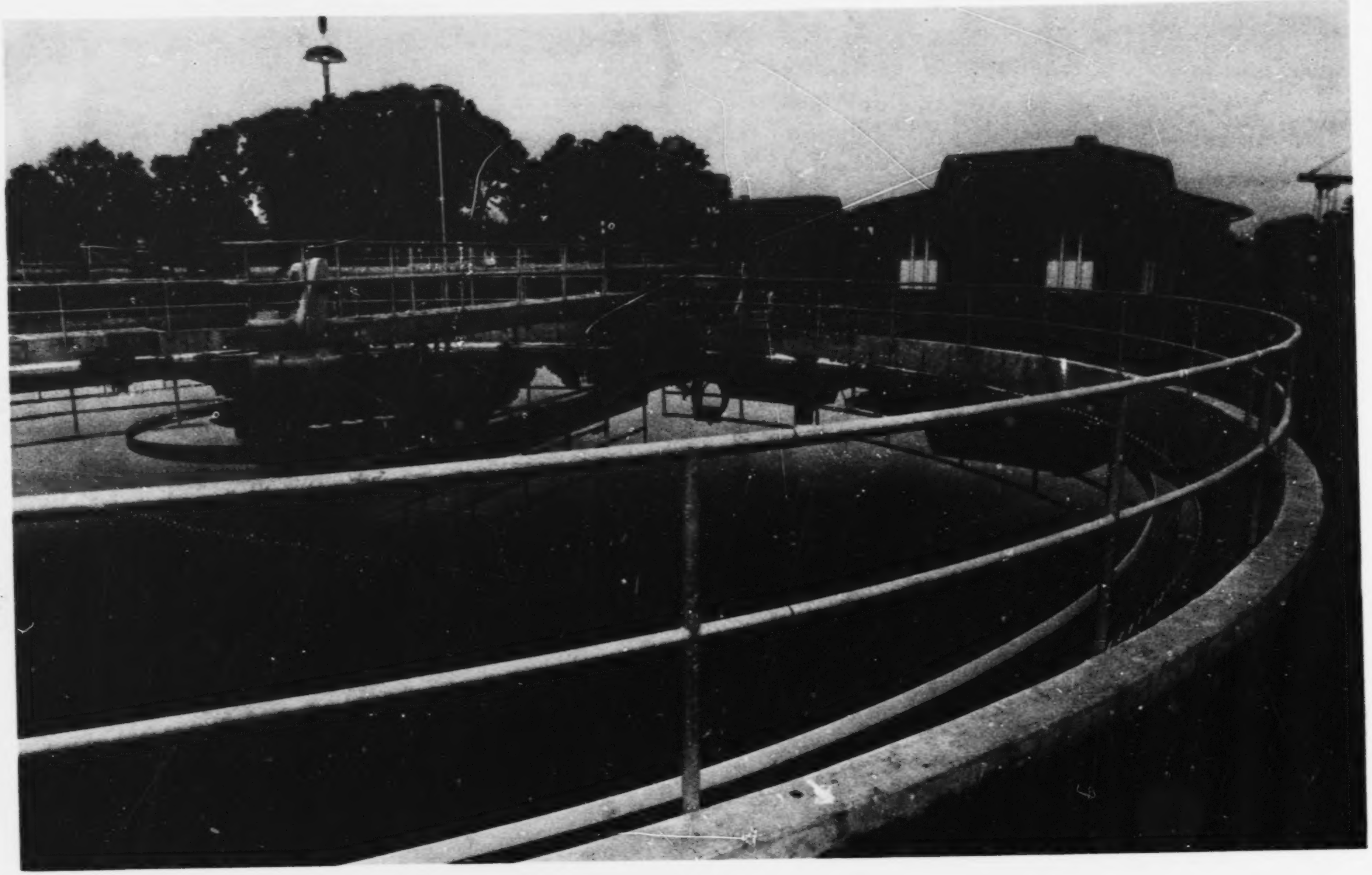

FIGURE A–1

CLARIFIER & CONTROL BUILDING

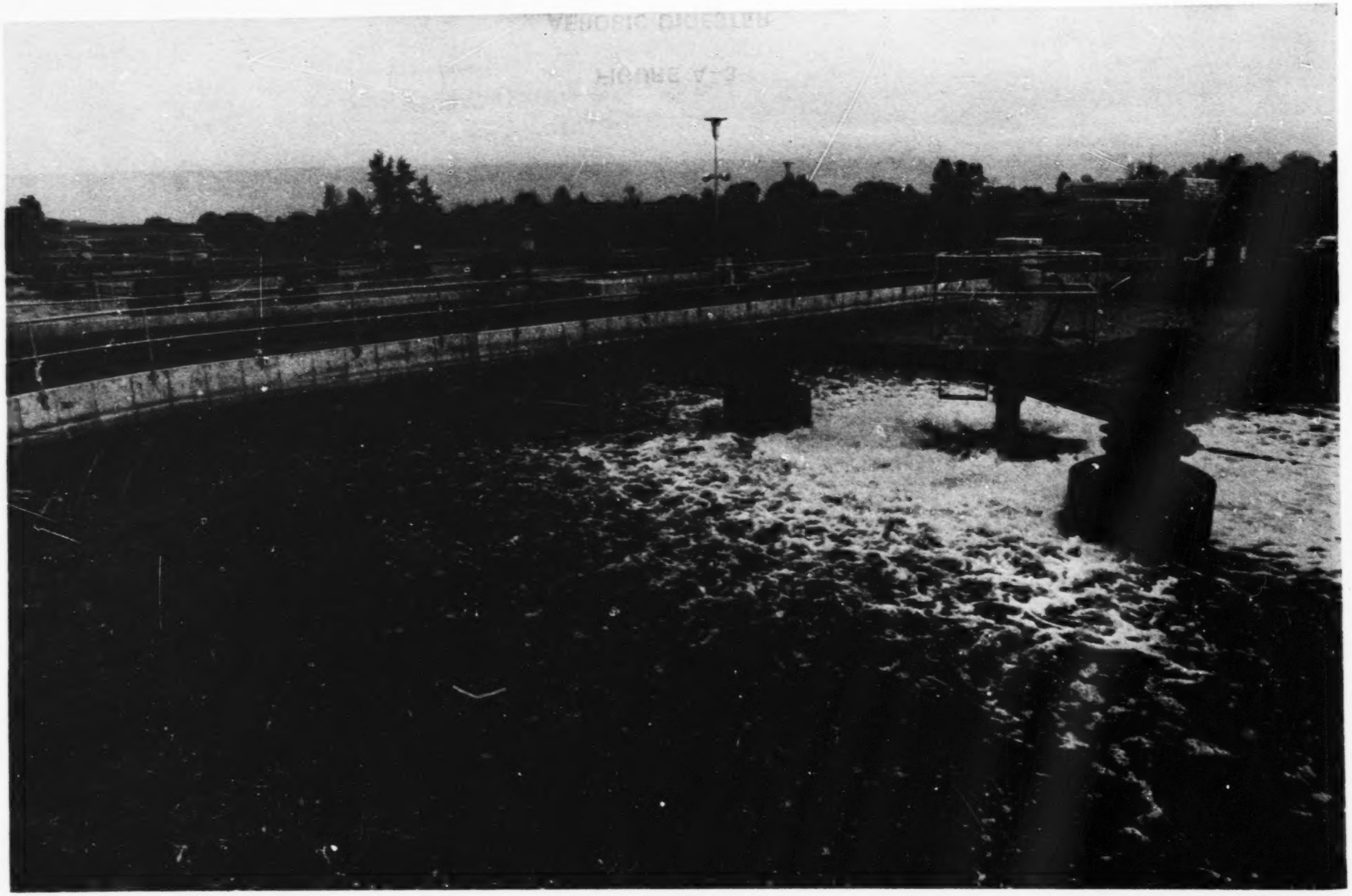

FIGURE A–2

AERATION BASINS

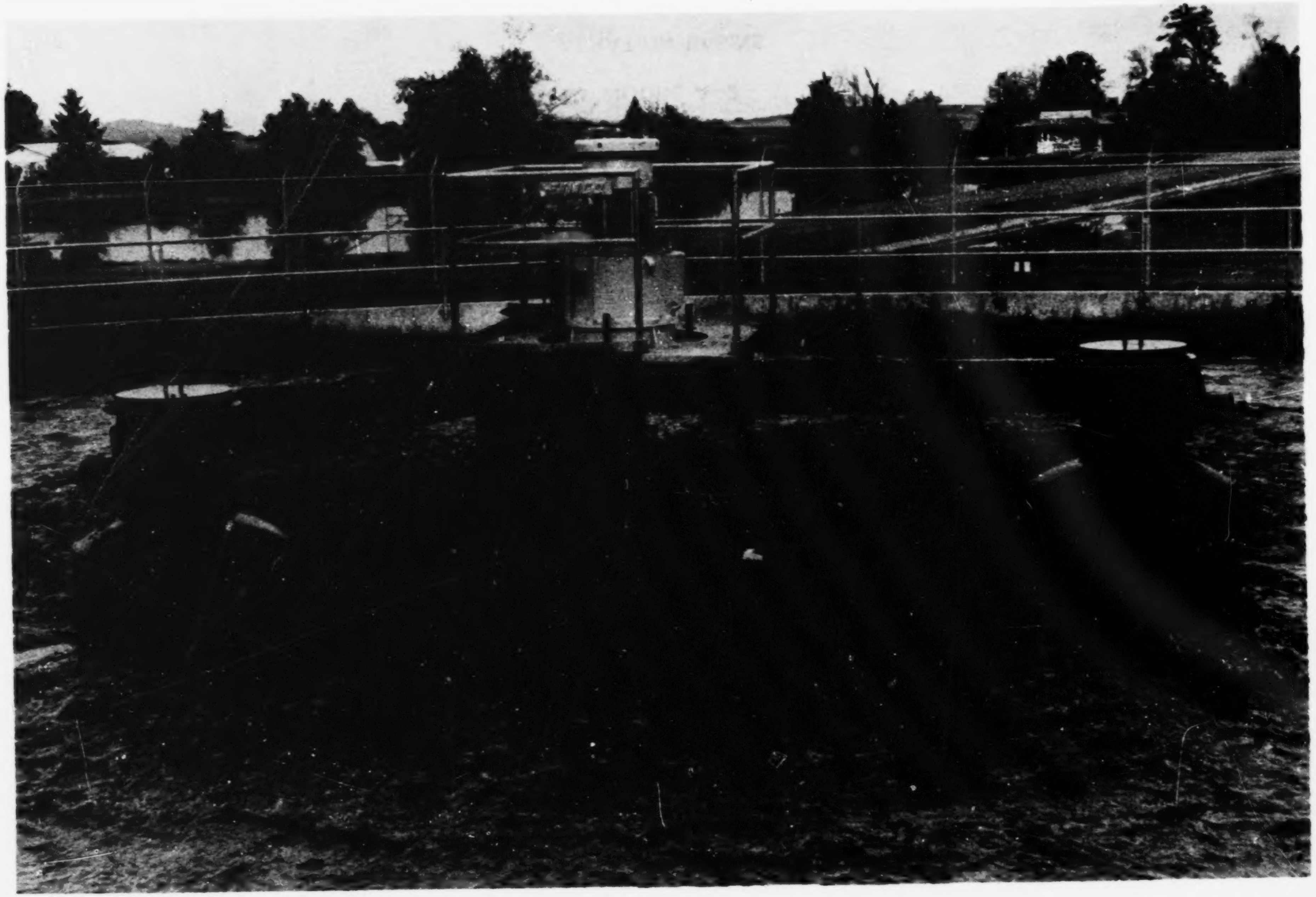

FIGURE A–3

AEROBIC DIGESTER

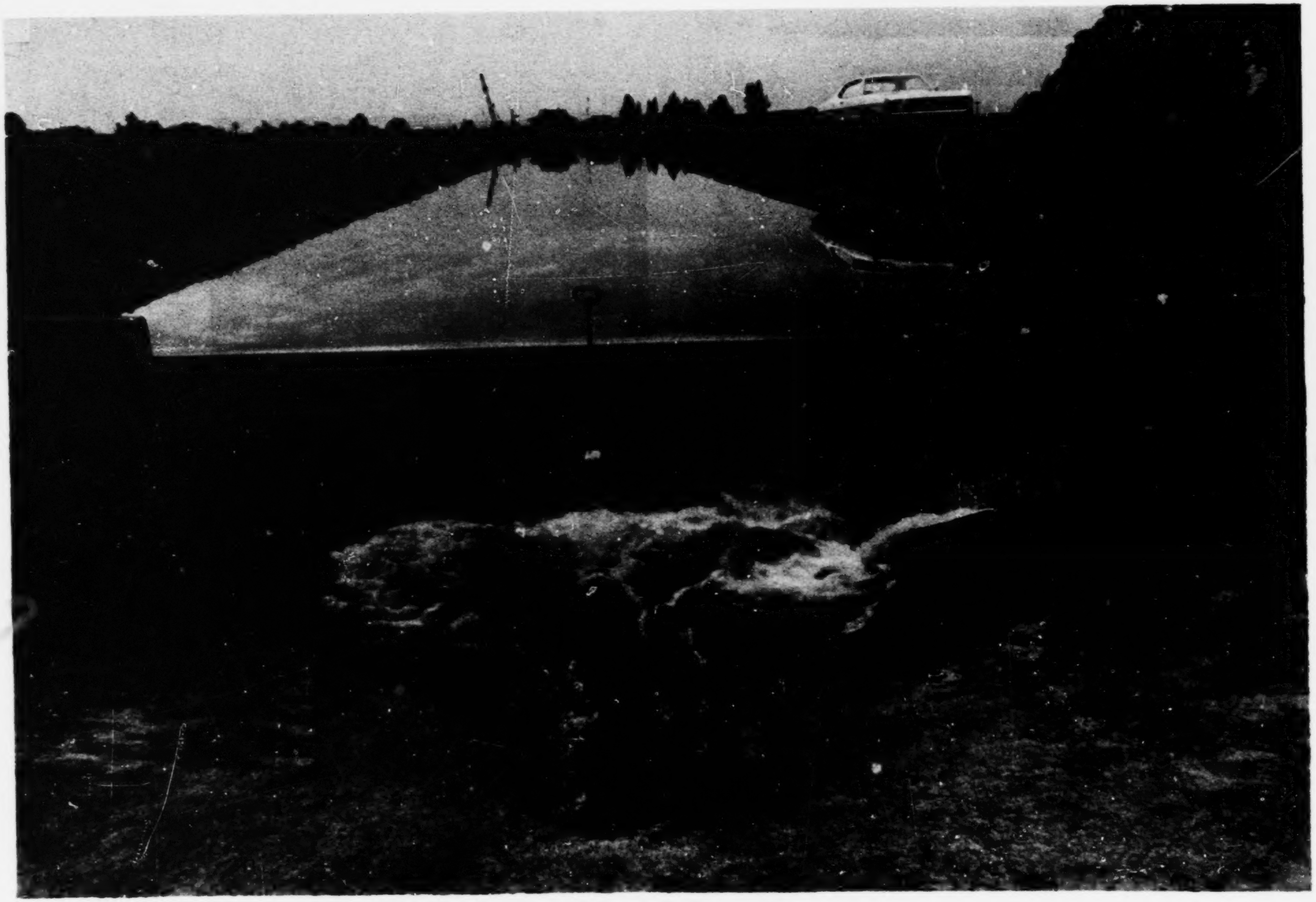

FIGURE A–4

CHLORINE CONTACT CHANNEL AND FINAL EFFLUENT

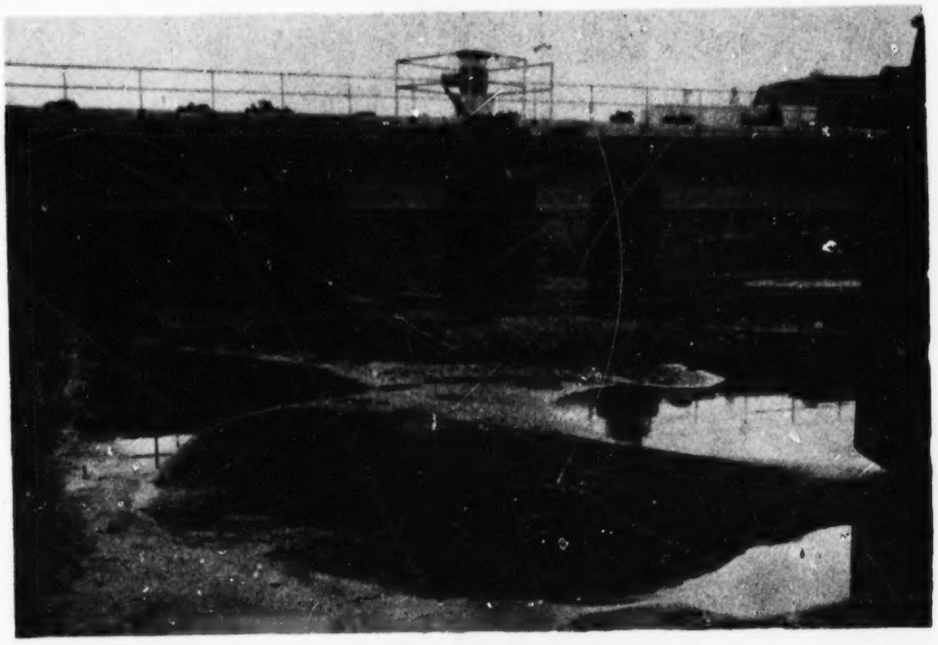

FIGURE A–5

SOLIDS DEPOSITS – AERATION BASIN NO. 1, N. SIDE

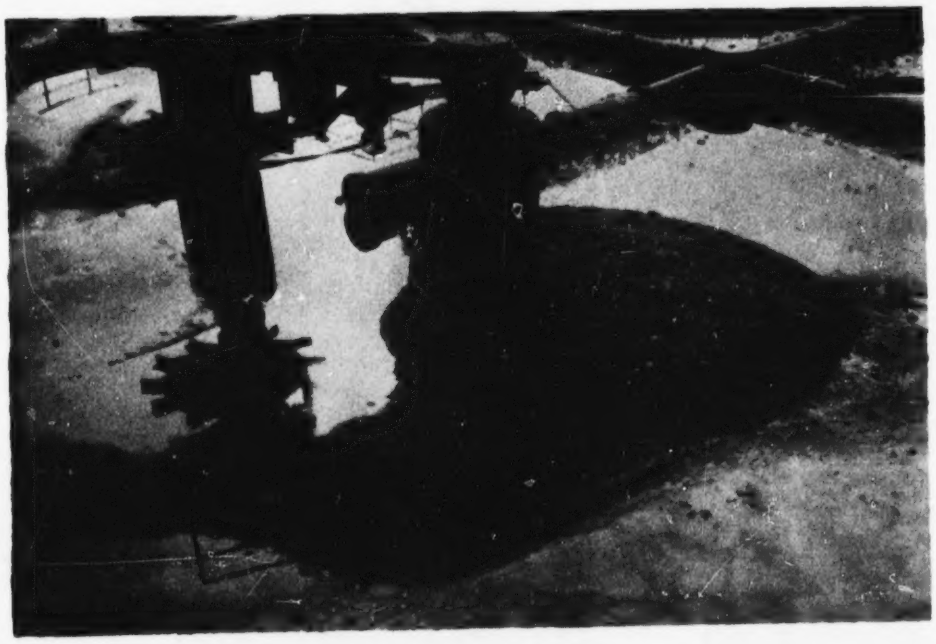

FIGURE A–6

SOLIDS DEPOSITS AND SCOUR–AERATION BASIN NO. 1

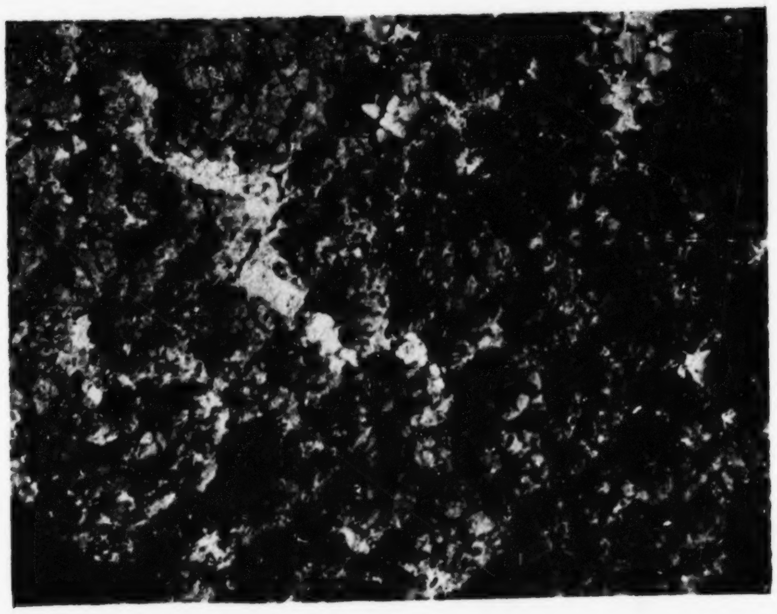

FIGURE A–7

DRIED HUMUS (2'x3' AREA)

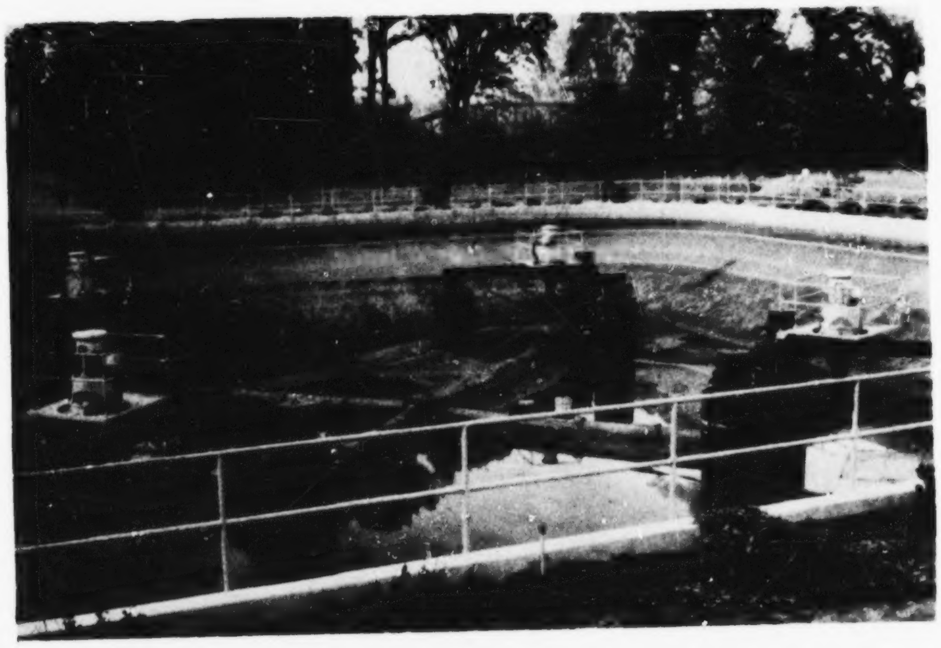

FIGURE A–8

AERATION BASIN NO. 1

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APPENDIX B

PROCESS AND LABORATORY EQUIPMENT

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PROCESS EQUIPMENT

EQUIPMENT	MODEL	MANUFACTURER
Comminutor	25A	Worthington Corp.
Sludge Recirculation Pumps	6FNJD-15	
Raw Waste Pumps	6FNC-15	
Aerators	Model 25 Model 30	Eimco Corp.
Clarifier	RSR Type	Dorr-Oliver, Inc.
Chlorination Equipment	V-800 Chlorinator	Wallace & Tiernan
Flowmeters	Propeller Type	Hersey-Sparling Meter Co.
Flowrecorder		
Sampler	260-B	
Waste Sludge Pumps	T4A3B	Gorman-Rupp Co.
Sample Pumps	FS-22	Robbins and Meyers Moyno Pump Division
Chlorinator Water Supply	PACO 1070-1	Pacific Pumping Co.
Nonpotable Water Pumps	PACO 1270-5	
Air Compressor		Bell & Gossett Products

LABORATORY EQUIPMENT

EQUIPMENT	TYPE	RANGE
Furnace	Thermolyne Type 1400	0 - 1400° C
Drying Oven	Precision Scientific	0 - 140° C
Magnetic Stirrer	VW & R Magnistir	
pH Meter	Photovolt	
Demineralizer	Barnstead Bantam	0 - 10 gal/hr.
D. O. Meter	YSI - Model 54	0 - 20 ppm
Balance	Voland & Sons Model 220D	0 - 200 g

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APPENDIX C

VELOCITY AND DISSOLVED OXYGEN DATA

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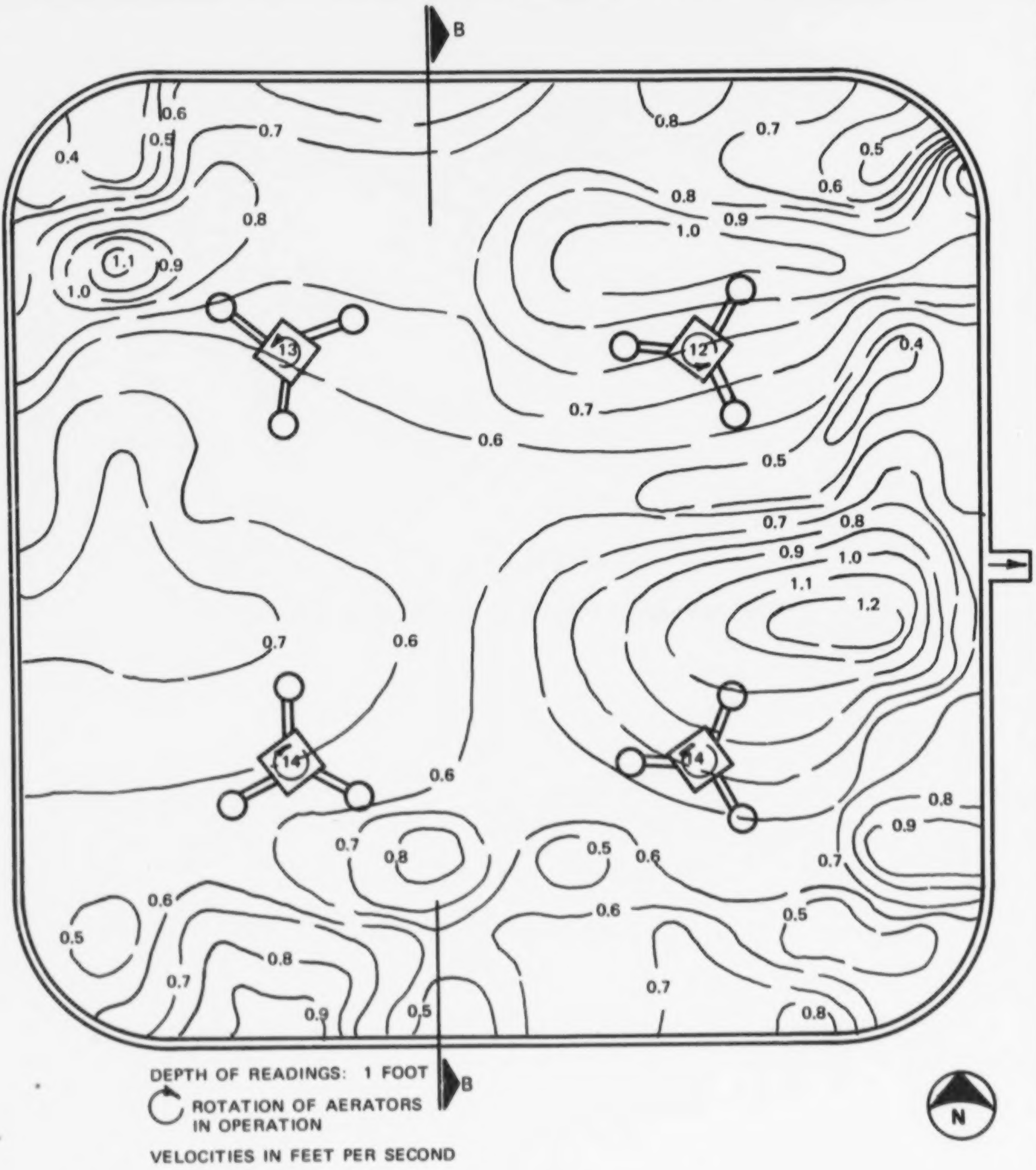
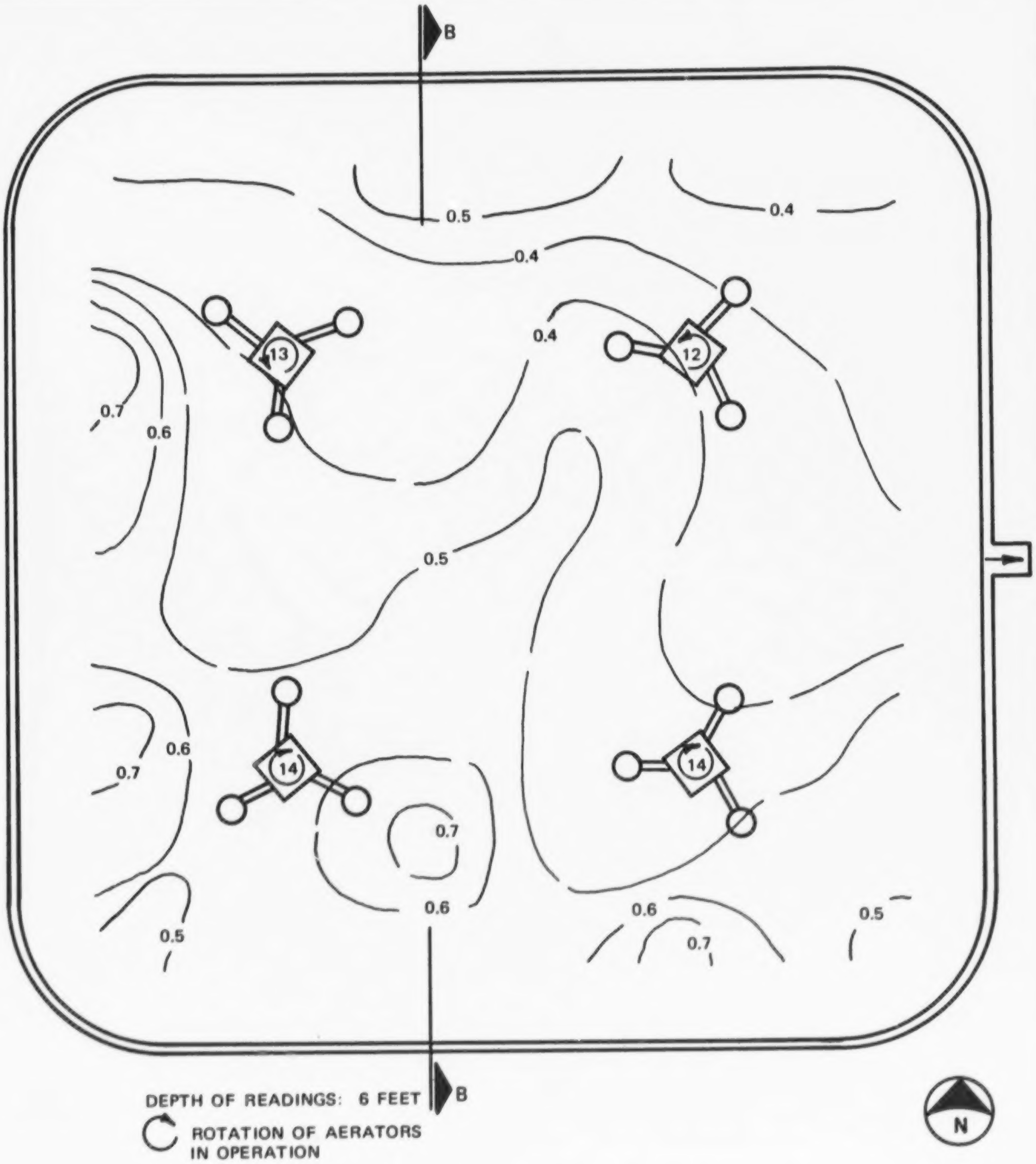

B
0.6
0.5
0.4
0.7
0.8
1.1
0.9
1.0
13
0.8
0.7
0.5
0.6
0.8
0.9
1.0
12
0.4
0.7
0.6
0.5
0.7
0.8
0.9
1.0
1.1
1.2
0.7
0.6
14
0.6
4
0.8
0.9
0.7
0.8
0.5
0.6
0.7
0.6
0.5
0.5
0.8
0.7
0.9
0.5
0.7
0.8
DEPTH OF READINGS: 1 FOOT
B
ROTATION OF AERATORS IN OPERATION
VELOCITIES IN FEET PER SECOND
N

FIGURE C-1

AERATION BASIN VELOCITY PROFILE
FOUR AERATORS – LOW SPEED


B
0.5
0.4
0.4
0.4
13
12
0.7
0.6
0.5
0.6
0.7
14
14
0.7
0.6
0.6
0.5
0.7
0.5
B
N

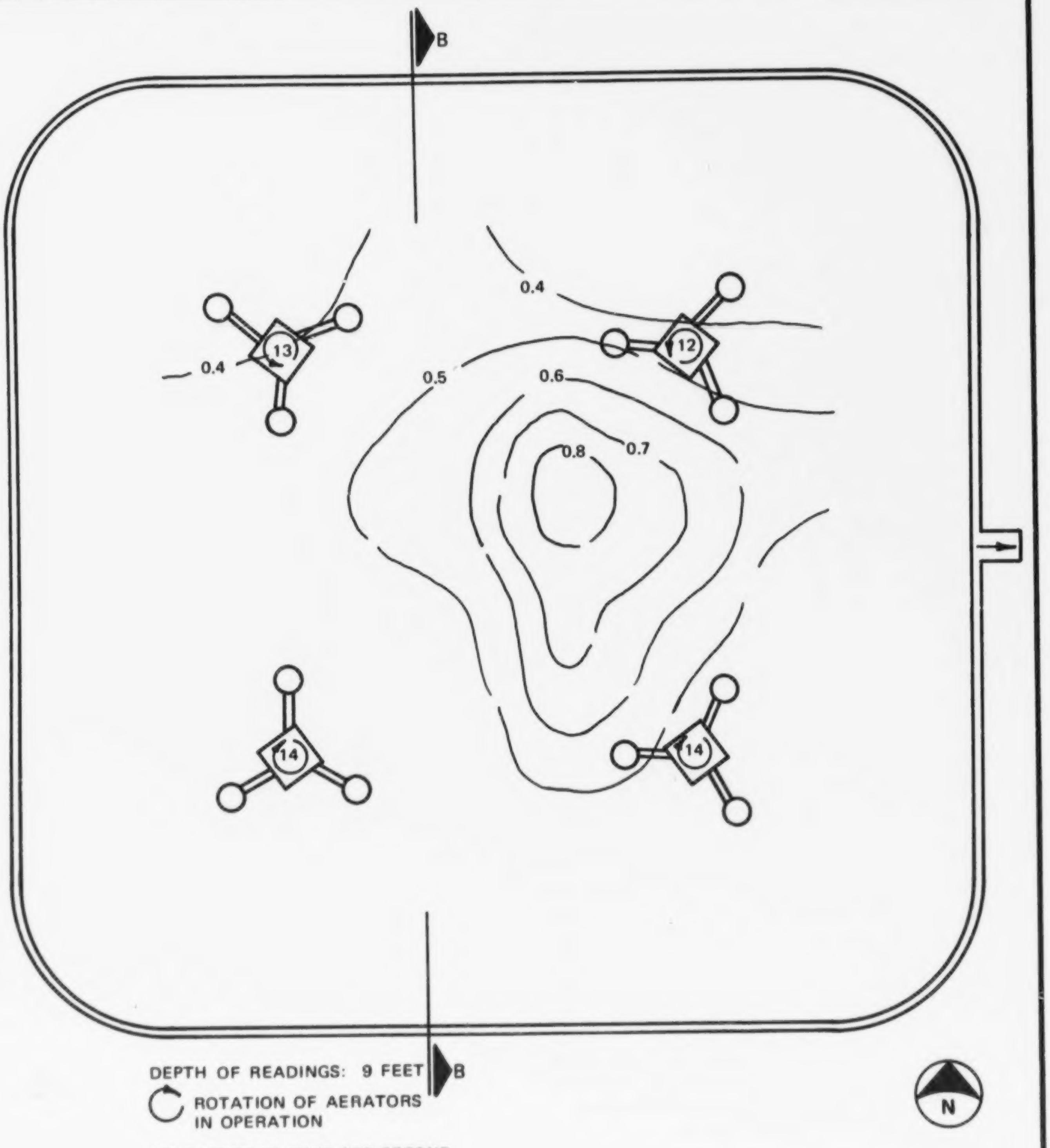
DEPTH OF READINGS: 6 FEET

ROTATION OF AERATORS IN OPERATION

VELOCITIES IN FEET PER SECOND

FIGURE C-2

AERATION BASIN VELOCITY PROFILE
FOUR AERATORS – LOW SPEED


B
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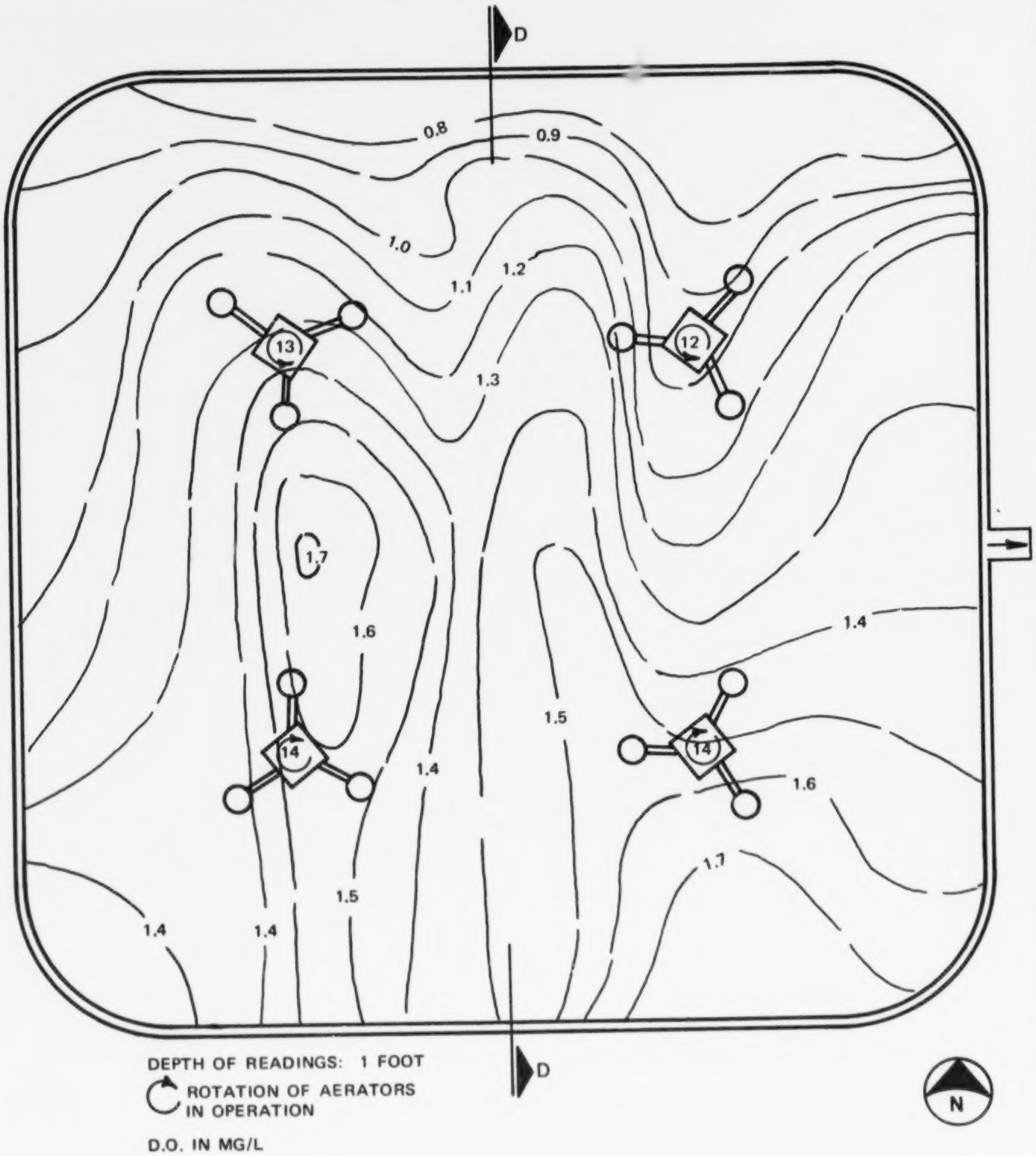
0.4
13
12
0.5
0.6
0.8
0.7
14
14
B
DEPTH OF READINGS: 9 FEET
ROTATION OF AERATORS IN OPERATION
VELOCITIES IN FEET PER SECOND
N

FIGURE C-3

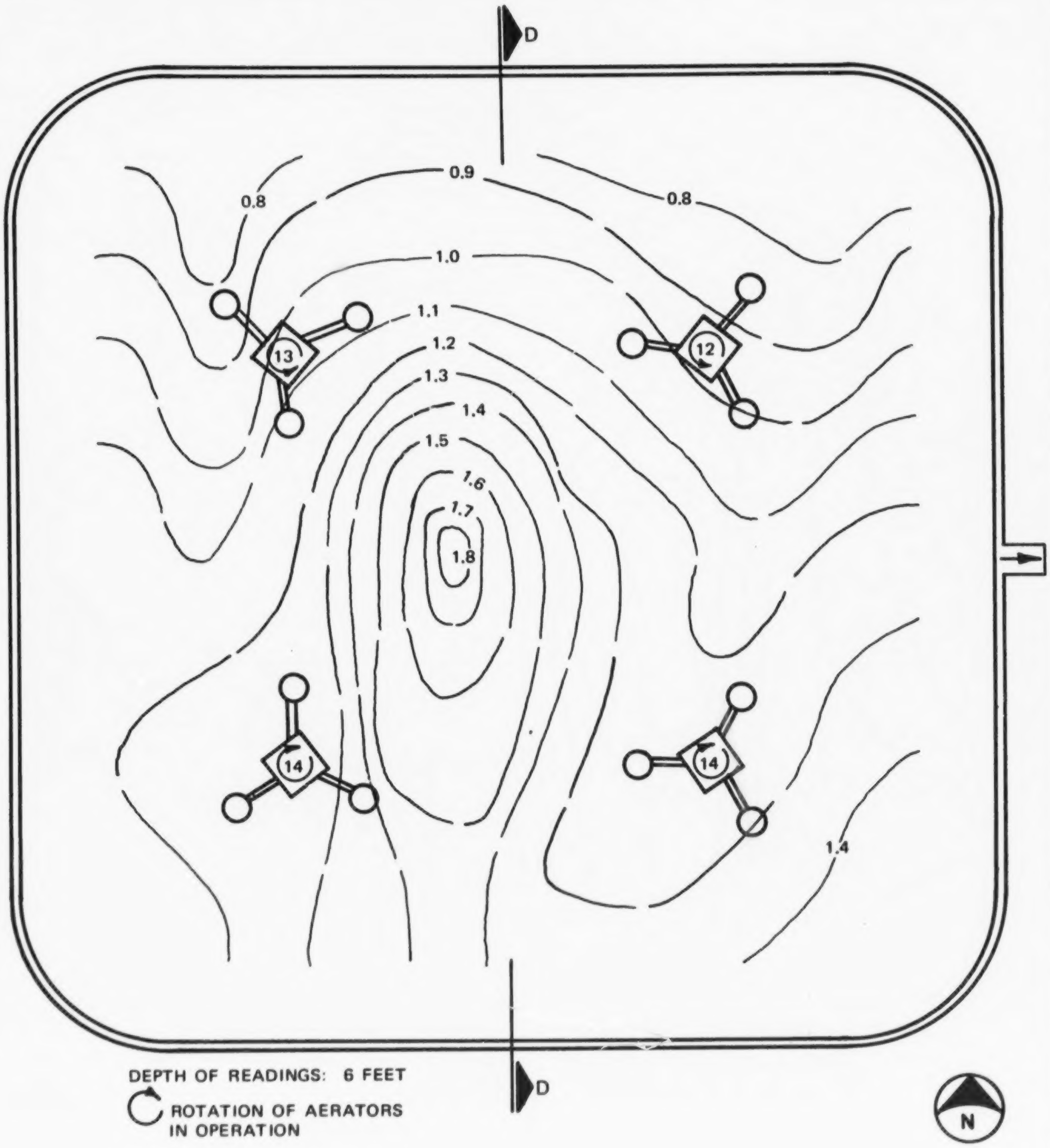
AERATION BASIN VELOCITY PROFILE
FOUR AERATORS – LOW SPEED


D
0.8
0.9
1.0
1.1
1.2
13
12
1.3
1.7
1.6
1.4
1.5
14
14
1.4
1.6
1.7
1.4
1.4
1.5
D
N

DEPTH OF READINGS: 1 FOOT
ROTATION OF AERATORS IN OPERATION
D.O. IN MG/L

FIGURE C-4

AERATION BASIN D.O. PROFILE
FOUR AERATORS – LOW SPEED


D
0.9
0.8
0.8
1.0
1.1
1.2
1.3
1.4
1.5
1.6
1.7
1.8
13
12
14
14
1.4
D
N

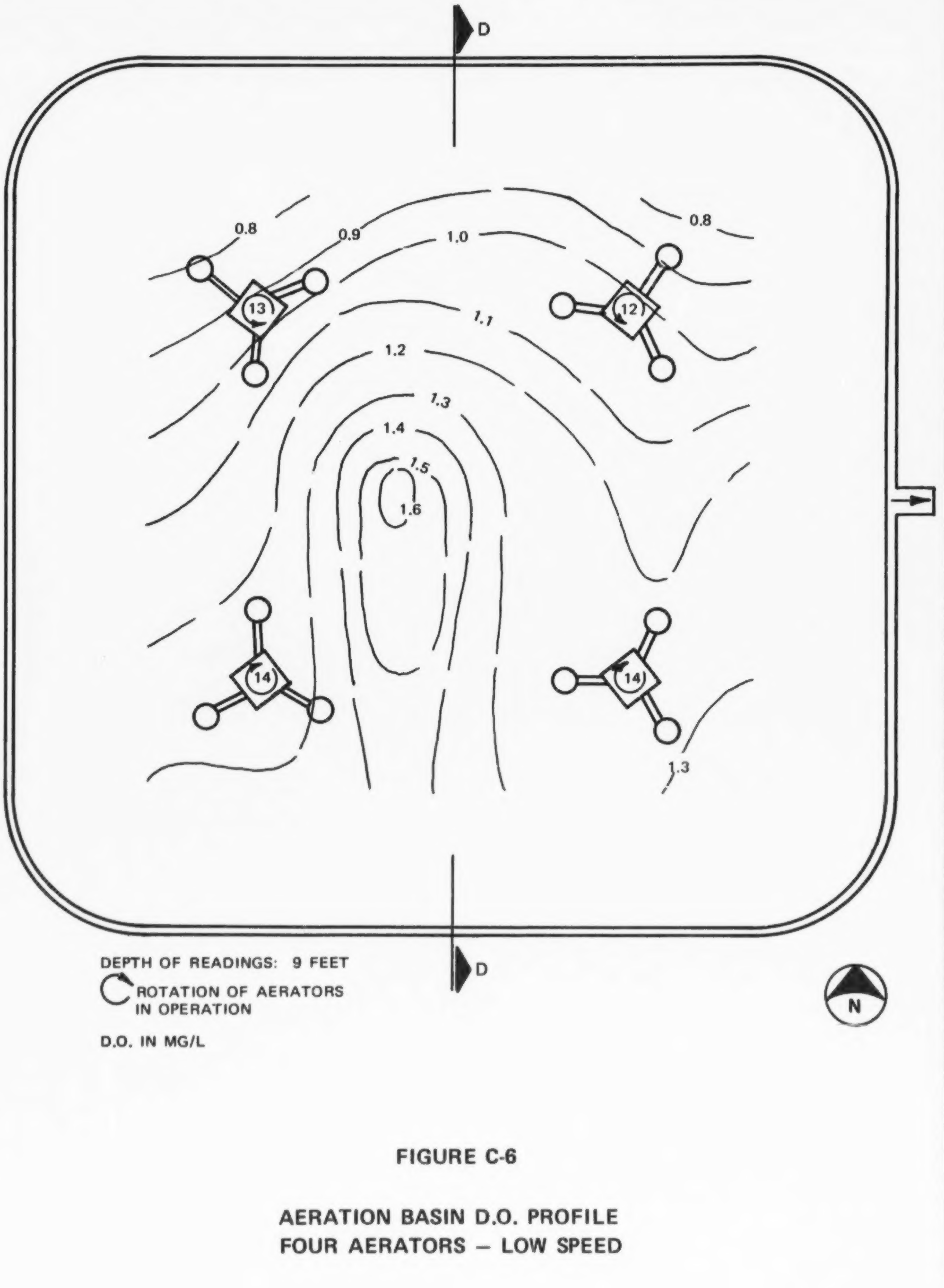
DEPTH OF READINGS: 6 FEET

ROTATION OF AERATORS IN OPERATION

D.O. IN MG/L

FIGURE C-5

AERATION BASIN D.O. PROFILE
FOUR AERATORS – LOW SPEED


D
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0.9
1.0
0.8
13
12
1.1
1.2
1.3
1.4
1.5
1.6
14
14
1.3
D
N

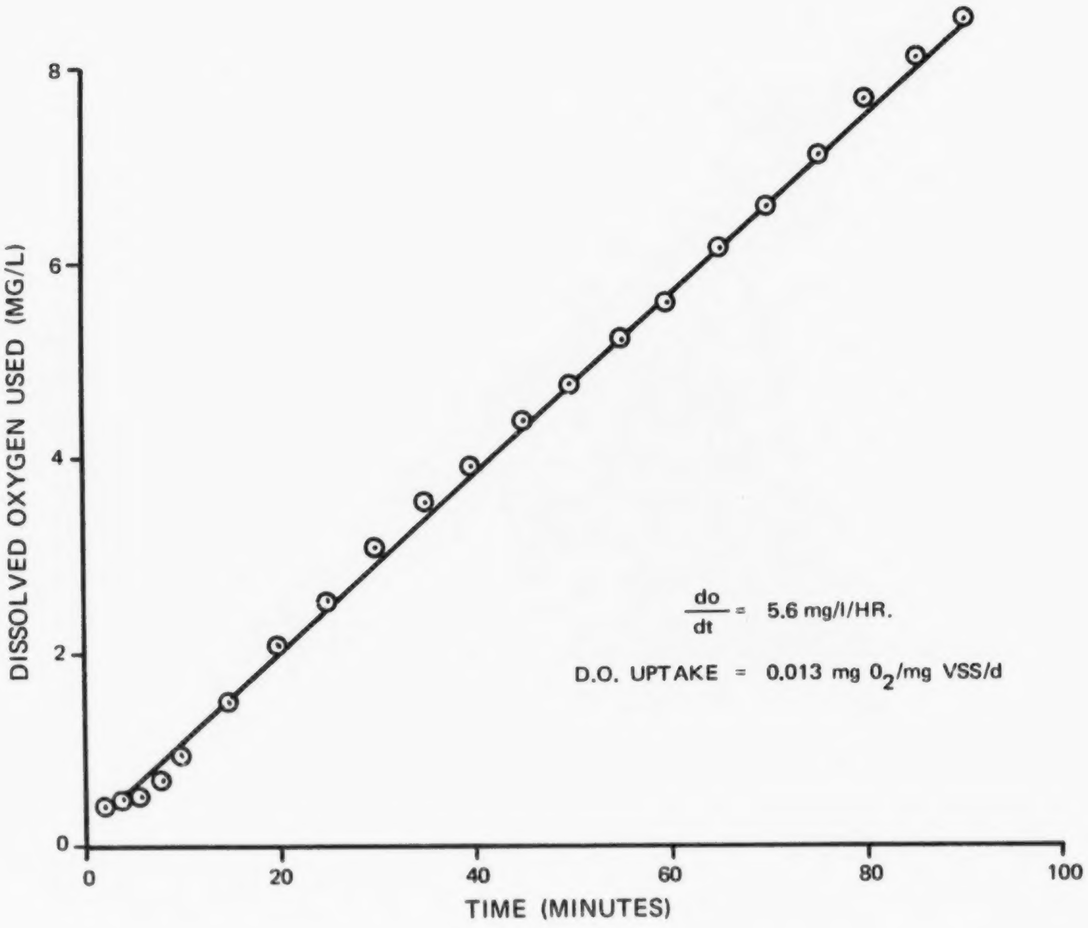
DEPTH OF READINGS: 9 FEET

ROTATION OF AERATORS IN OPERATION

D.O. IN MG/L

FIGURE C-6

AERATION BASIN D.O. PROFILE
FOUR AERATORS – LOW SPEED



$\frac{do}{dt}$ = 5.6 mg/l/HR.

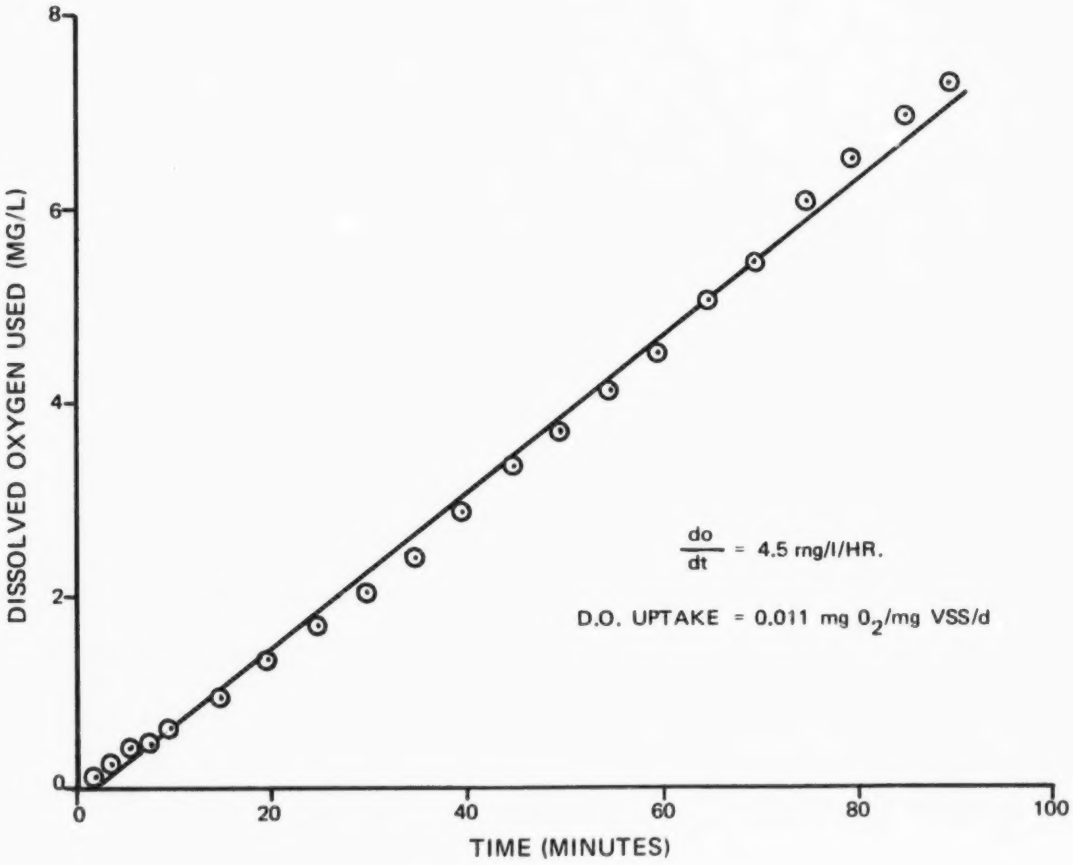
D.O. UPTAKE = 0.013 mg 0_2/mg VSS/d

DATE: OCT. 16, 1970
TIME: 10:00 A.M.
TEMP.: 11°C

TSS = 15,000 mg/l
VSS = 10,000 mg/l

FIGURE C-7

DISSOLVED OXYGEN DATA
DIGESTER



DATE: OCT. 16, 1970

TIME: 1:15 P.M.

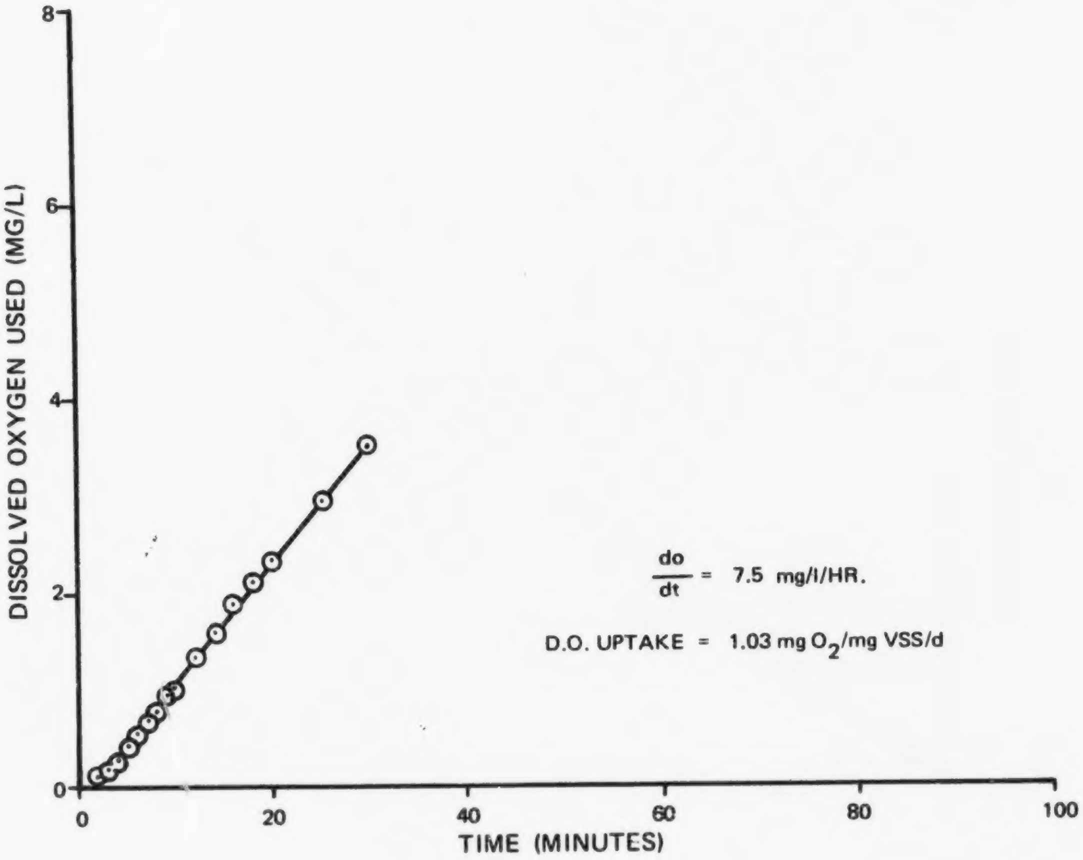
TEMP.: 12.5°C

TSS = 15,000 mg/l

VSS = 10,000 mg/l

FIGURE C-8

DISSOLVED OXYGEN DATA
DIGESTER



$\frac{do}{dt}$ = 7.5 mg/l/HR.

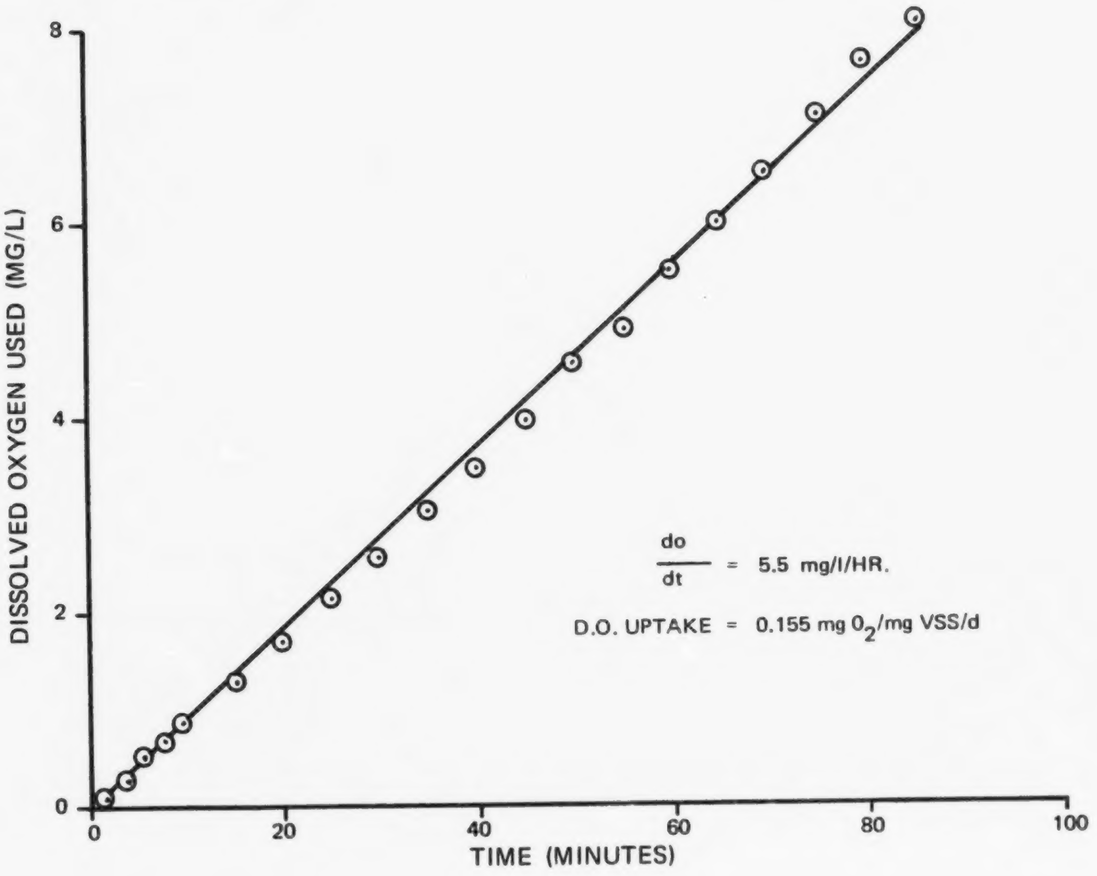
D.O. UPTAKE = 1.03 mg O_2/mg VSS/d

DATE: OCT. 14, 1970
TIME: 11:30 AM
TEMP: 19°C

TSS = 630 mg/l
VSS = 175 mg/l

FIGURE C-9

OXYGEN UPTAKE DATA
AERATION BASIN



$\frac{do}{dt}$ = 5.5 mg/l/HR.

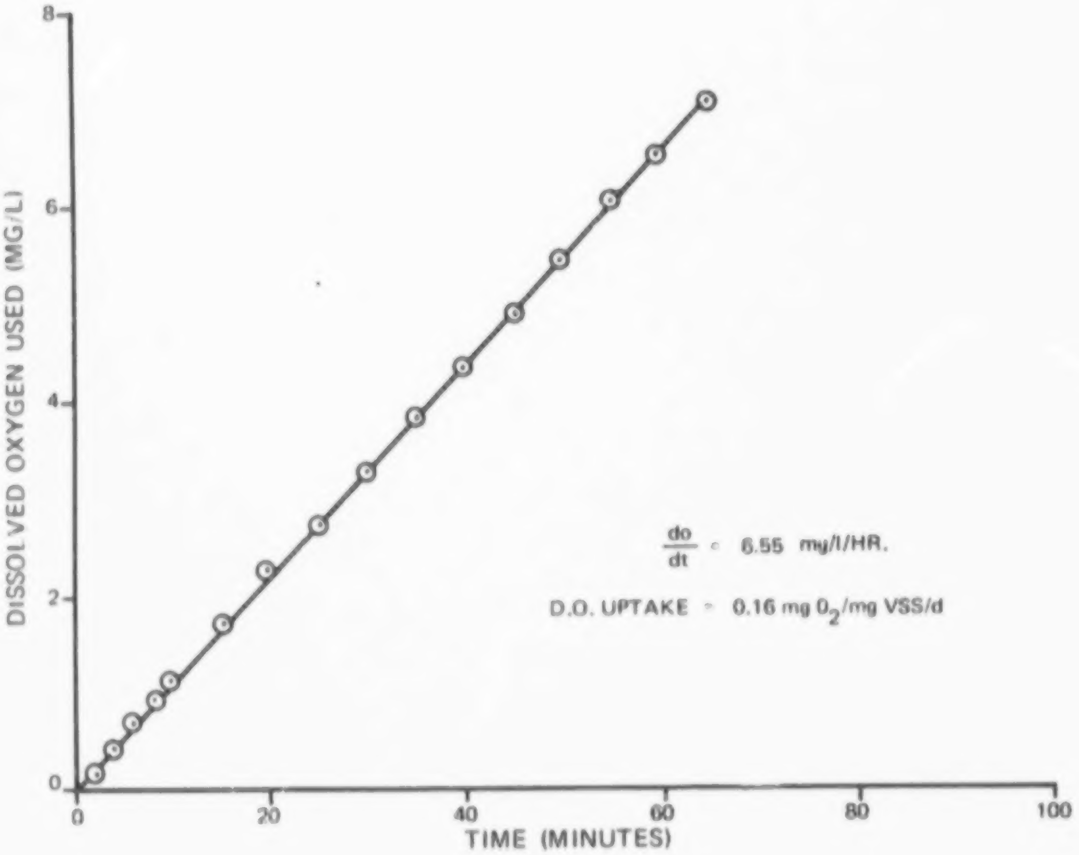
D.O. UPTAKE = 0.155 mg 0_2/mg VSS/d

DATE: OCT. 15, 1970
TIME: 9:30 A.M.
TEMP.: 17.5°C

TSS = 1360 mg/l
VSS = 850 mg/l

FIGURE C-10

DISSOLVED OXYGEN DATA
AERATION BASIN



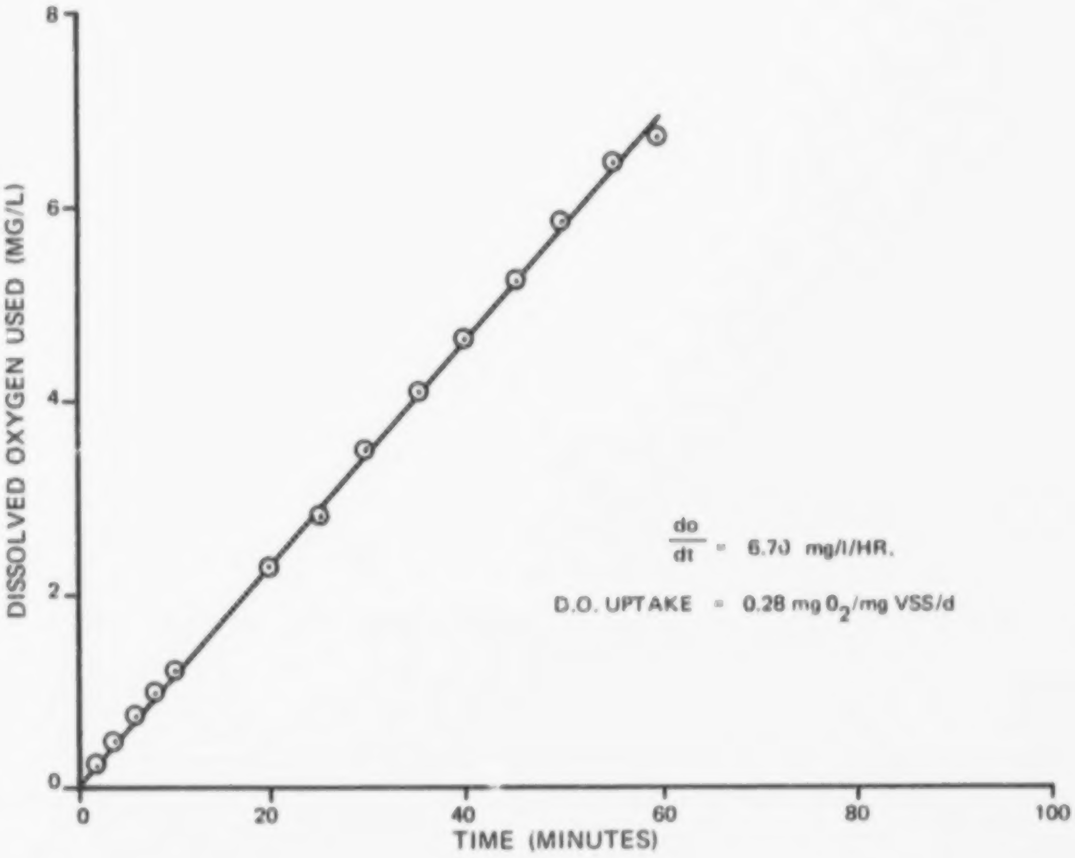
$\frac{do}{dt}$ = 6.55 mg/l/HR.

D.O. UPTAKE = 0.16 mg O_2/mg VSS/d

DATE:	OCT. 15, 1970	TSS = 1240 mg/l	
TIME:	11:00 A.M.	VSS = 990 mg/l	
TEMP.:	17.5°C		

FIGURE C-11

DISSOLVED OXYGEN DATA
AERATION BASIN



$\frac{do}{dt}$ = 6.70 mg/l/HR.

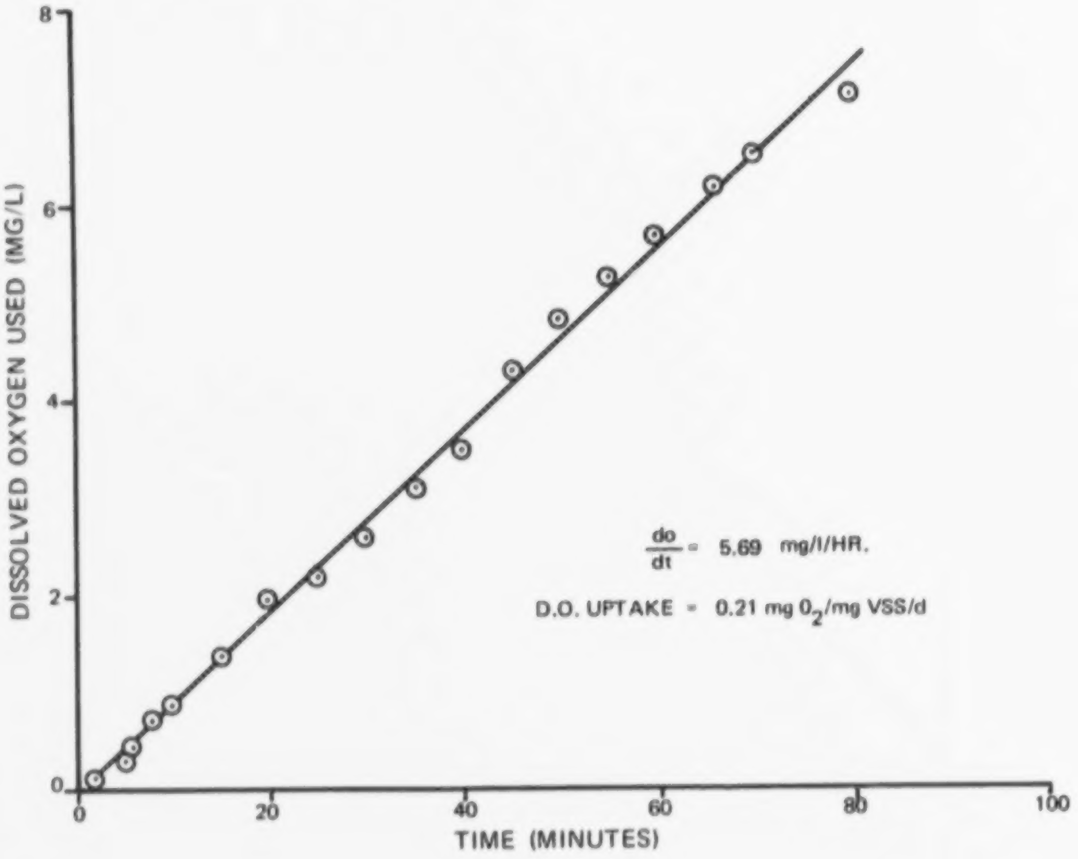
D.O. UPTAKE = 0.28 mg 0_2/mg VSS/d

DATE: OCT. 15, 1970
TIME: 1:45 P.M.
TEMP.: 17.5°C

TSS = 850 mg/l
VSS = 570 mg/l

FIGURE C-12

DISSOLVED OXYGEN DATA
AERATION BASIN



$\frac{do}{dt}$ = 5.69 mg/l/HR.

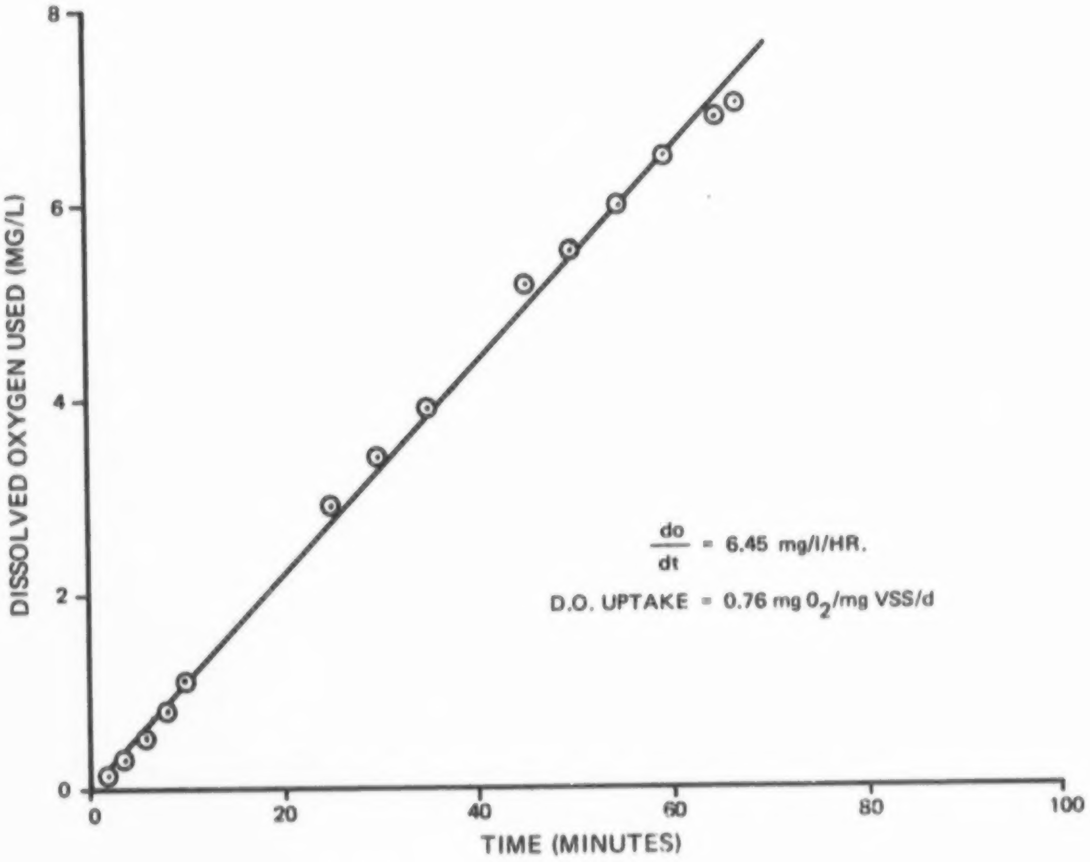
D.O. UPTAKE = 0.21 mg 0_2/mg VSS/d

DATE: OCT. 14, 1970
TIME: 4:00 P.M.
TEMP: 19°C

TSS = 860 mg/l
VSS = 660 mg/l

FIGURE C-13

DISSOLVED OXYGEN DATA
AERATION BASIN



DATE: OCT. 14, 1970 TSS = 680 mg/l

TIME: 1:30 P.M. VSS = 205 mg/l

TEMP.: 19°C

FIGURE C-14

OXYGEN UPTAKE DATA
AERATION BASIN

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APPENDIX D

COSTS

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TABLE D-1
CITY OF DALLAS, OREGON
WATER POLLUTION CONTROL PLANT
CONSTRUCTION COSTS

ITEM	TOTAL COST
Temporary Facilities	$ 12,000
Bond and Insurance	8,000
Move-In and Site Preparation	20,000
Cleanup and Finish Grading	2,000
Earthwork	7,000
Headworks	4,000
Aeration Basin Splitter Box	3,500
Aeration Basins	83,000
Clarifier and Sludge Recirculation Pump Station	40,600
Aerobic Digester	19,600
Flow Measurement Box	600
Chlorine Contact Channel	12,500
Humus Ponds	8,000
Control Building	23,000
Maintenance and Storage Building	1,500
A.C. Road and Parking	7,250
Gravel Road and Parking	2,000
Concrete Walks and Curbs	1,800
Fencing	7,000
Painting	6,000
Outside Piping and Plumbing	30,500
Sewage Shredder	10,000
Aeration Equipment	81,000
Clarifier Mechanism	17,500
Sludge Recirculation Pumps	9,500
Self Priming Sludge Pumps	2,150
Flow Measurement Equipment	4,000
Chlorination Equipment	6,100
Miscellaneous Pumps and Sampler	2,000
Hoisting Equipment	2,500
Laboratory Equipment and Furnishings	3,000
Electrical	21,000
Raw Sewage Pump Station	73,000

TABLE D-1 · Continued

ITEM	TOTAL COST
Plans and Specifications	$ 31,480
Test Borings	400
Supervision and Inspection of Construction	20,910
Administrative, Legal and Fiscal Costs	1,080
Land	20,000
TOTAL	$605,470

TABLE D-2
DEMONSTRATION, OPERATION AND MAINTENANCE COSTS

ITEM	OPERATION AND MAINTENANCE COSTS	DEMONSTRATION COSTS
Labor	$11,460.87	
Electrical Power	6,395.72	
Chemicals	2,153.39	
Maintenance	956.99	
Miscellaneous (Supplies, etc.)	1,225.26	
Post Construction Studies and Reports		$ 8,319.91
Legal and Fiscal Costs		506.45
Administrative Costs		1,679.19
TOTALS	$22,192.23	$10,505.55

Total Cost - First Year Demonstration - $32,697.78

APPENDIX E

RESULTS OF PVC TESTS

OREGON STATE UNIVERSITY
CORVALLIS, OREGON
SCHOOL OF ENGINEERING
Reply to: DEPARTMENT OF
GENERAL ENGINEERING

December 8, 1970

Mr. Robert Burm, Staff Engineer
National Waste Treatment Research Program
Pacific Northwest Water Laboratory
200 S.W. 35th St.
Corvallis, Oregon 97330

Dear Mr. Burm:

Following are my interpretations of the test results on the PVC liner. I have assumed that the used material (A) was the same as the 20 mil new material (B) initially.

There is evidently some loss in material over the one year use as a sewage pond liner. The chemical action, however, is apparently not detrimental to the properties. All measured properties show an increase in resistance to load: approximately 10% increase in tensile strength and secant modulus and about 20% increase in tear resistance. The percent elongation at break shows about a 20% increase, also. The Change in all these factors can be conveniently expressed as a significant increase in toughness.

The one other significant result is related to the standard deviations. The standard deviations of both the new samples are essentially the same while those of the used sample are approximately 50% to 200% larger than those of the new samples. As could have been foreseen, this indicates that the effect of the chemical environment is quite variable even though the trend in change is as indicated in the previous paragraph.

One would expect to approach optimum use time after which the properties as measured in these tests would begin to decrease and at some later time come back to the original values and thence decrease further. The determination of the regression time to original values is important as it may be anywhere from relatively short to quite long. Ultimately, the material may fail as a result of extreme thinning (with the occurance of holes) before decreased strength values leading to failure occur.

If you should require any further assistance, please feel free to call upon me.

Yours truly,

David A. Bucy

David A. Bucy
Associate Prof.

OSU
CORVALLIS, OREGON 97331

OREGON STATE UNIVERSITY
DEPARTMENT OF
METALLURGICAL ENGINEERING
Graf Hall 206
Area Code 503: 754-2654

December 1, 1970

Mr. Robert Burm, Staff Engineer
National Waste Treatment Research Program
Pacific Northwest Water Laboratory
200 S.W. 35th St.
Corvallis, Oregon 97330

Dear Mr. Burm:

Enclosed are the results of the tests you requested on the three samples of plastic liner. Also included is a detailed procedure followed during testing.

As discussed by telephone on October 20, 1970, the results of these tests are mutually comparable and should be comparable to further tests conducted following the outlined procedure. The tear test results are not necessarily comparable to tests conducted following ASTM D1004 because of the lack of a die for cutting the samples. Other than the lack of a die, the tests were conducted following ASTM D882 and ASTM D1004.

If you should require any further assistance, please feel free to call upon me.

Yours truly,

David A. Bucy

David A. Bucy
Associate Professor

DAB:jaw

SUMMARY OF TESTS

	A	B	C
Gage Length (Tensile)	5	5	5
Nominal Width, in.	1	1	0.5
Nominal Thickness, in.	0.0195	0.0209	0.0299
Break Factor, lb/in.	40.5	40.2	64.5
S	2.2	1.1	1.4
Tensile Strength, psi.	2090	1920	2170
S	110	48	43
% Elongation at Break	238	195	304
S	31	9.7	10
Secant Modulus of Elasticity, ΔL = 3 in., psi	1760	1600	1290
S	55	38	35
Tear Load, lb	7.88	6.51	9.15
S	0.74	0.41	0.66

A - used 20 mil PVC

B - new 20 mil PVC

C - new 30 mil PVC

PLASTIC SHEETING TEST REPORT

FOR: Mr. Robert Burm, Pacific Northwest Water Laboratory

BY: David A. Bucy, P.E., State of Oregon, Department of Metallurgical Engineering, Oregon State University

Procedures for Testing Plastic Sheeting

Tensile Properties Test

ASTM D882 (copy appended) was followed. Report proper includes all pertinent information.

Tear Resistance Test

ASTM D1004 (copy appended) was followed except for the die described under paragraph 3.(e). Samples were cut through a mimeographed copy of the shape attached to the material using a razor blade. Particular attention was paid to the 90° notch to ensure no over cut occurred.

TENSILE PROPERTIES OF THIN PLASTIC SHEETING

1. Material

Three lots of material were supplied by Robert Burm, Pacific Northwest Water Laboratory and will be designated A,B, and C as follows:

A. Used plastic liner. Apparently, from cutting, density, and burn test, it is polyvinyl chloride (PVC). Assumed to be isotropic in plane of sheet.* Six strips approximately 3 inches by 20 inches.

B. New sheet approximately 13 inches by 32 inches. Marked "20 mil thickness, PVC" by R. Burm. Thought to be identical with A but unused. Cutting, density, and burn test results same as A (PVC).

C. New sheet approximately 17 inches by 29 inches. Thicker than A and B. Cutting, density, and burn test results same as A and B (PVC). Assumed to be isotropic in plane of sheet.*

2. Preparation of Test Specimens

Test specimens were cut with a razor blade moved along a straight edge guide. Specimen edges were inspected microscopically to detect nicks or tears.

3. Specimen Dimensions (Nominal)

Material	Thickness	Width	Length
A	0.0195 in.	1.0 in.	7.0 in.
B	0.0209 in.	1.0 in.	7.0 in.
C	0.0299 in.	~~1.0~~ 0.5 in. ACC3	7.0 in.

---

* All sample pieces are too small to identify original roll transverse and longitudinal directions.

4. Number of specimens

   Six specimens of each material were tested.

5. Strain rate - 0.1 in./in./min. was used in order to meet requirement for Modulus of Elasticity measurement.

6. Initial grip separation - 5 inches.

7. Crosshead speed - 0.5 in./min.

9. Grips - rubber faced approximately 1 inch x 1 inch square as supplied by Instron.

10. Test method - A

11. Conditioning
    Temperature - 70°F
    Relative Humidity -

12. Anomalous behavior - Tear in one specimen.

1 *Accession Number*

2 *Subject Field & Group*
05D

SELECTED WATER RESOURCES ABSTRACTS
INPUT TRANSACTION FORM

5 *Organization*
Dallas, Oregon, City of

6 *Title*
Combined Treatment of Domestic and Industrial Wastes
By Activated Sludge

10 *Author(s)*
Dallas, Oregon,
City of

16 *Project Designation*
Grant No. 12130 EZR (11060 EZR)

21 *Note*

22 *Citation*

23 *Descriptors (Starred First)*
*Waste treatment, *Aerobic treatment, Industrial wastes,
Activated Sludge, Domestic Wastes

25 *Identifiers (Starred First)*
*Food processing wastes, Aerobic digestion, Treatment costs.

27 *Abstract*

The operation of a completely aerobic secondary treatment facility for treatment of combined domestic and industrial wastewater from the City of Dallas, Oregon, was studied for a period of 15 months. The system was designed for an average daily flow of 2.0 mgd and a BOD load of 7000 pounds per day. The results of this study indicate the flexibility and economy of the completely aerobic system, consisting of activated sludge with aerobic digestion, for a small community with proportionately high industrial wastewater loads. The effluent BOD concentration averaged 8 mg/l and the effluent total suspended solids concentration averaged 13 mg/l for the 15-month study period. The biological solids yield averaged about 0.7 pounds of solids per pound of BOD removed and the net accumulation of biological volatile solids was about 0.42 pounds of volatile solids per pound of BOD removed. These values were obtained with a MLSS concentration range of 700 to 3000 mg/l, an average sludge age of 19 days and an organic loading range of 0.05 to 0.40 pounds of BOD per pound of MLSS per day. Total capital cost of the system was about 66 percent of that for a conventional activated sludge plant and operation and maintenance costs were only about 33 percent of those for a conventional system.

This report was submitted in fulfillment of Grant No. 11060 EZR under the partial sponsorship of the Water Quality Office, Environmental Protection Agency.

Abstractor John L. Graham
Institution Cornell, Howland, Hayes & Merryfield, Corvallis, Ore.

WR:102 (REV. JULY 1969)
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